TE
662
.A3
no.
FHWA-
RD-
74-57

Dept. Of Transportation
JAN 22 1976
Library

port No. FHWA-RD-74-57

ROCEEDINGS OF WORKSHOP ON CUT-AND-COVER NNELING: PRECAST AND CAST-IN-PLACE DIAPHRAGM ALLS CONSTRUCTED USING SLURRY TRENCH TECHNIQUES

D. J. D'Appolonia, E. D'Appolonia, and D. Namy

January 1974
Final Report

This document is available to the public
through the National Technical Information
Service, Springfield, Virginia 22151

Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

NOTICE

This document is disseminated under the sponsorship of the Department of Transportation in the interest of information exchange. The United States Government assumes no liability for its contents or use thereof.

The contents of this report reflect the views of the contracting organization, which is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policy of the Department of Transportation. This report does not constitute a standard, specification, or regulation.

TE
662
.A3
no.
FHWA-
RD-
74-57

Dept. Of Transportation
JAN 22 1975
Library

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWA-RD-74-57	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle
Proceedings of Workshop on Cut-and-Cover Tunneling: Precast and Cast-in-Place Diaphragm Walls Constructed Using Slurry Trench Techniques

5. Report Date
January 1974

6. Performing Organization Code

7. Author(s)
David J. D'Appolonia, E. D'Appolonia and Dominique Namy

8. Performing Organization Report No.

9. Performing Organization Name and Address
ECI-Soletanche, Inc.
10 Duff Road
Pittsburgh, Pennsylvania 15235

10. Work Unit No.

11. Contract or Grant No.

12. Sponsoring Agency Name and Address
Offices of Research and Development
Federal Highway Administration
U.S. Department of Transportation
Washington, D.C. 20590

13. Type of Report and Period Covered
Final Report

14. Sponsoring Agency Code

15. Supplementary Notes
The workshop was sponsored jointly by the Office of the Secretary of Transportation and the Federal Highway Administration. Contract Manager: S. I. Majtenyi; Implementation Manager: David Nelson.

16. Abstract

The workshop, conducted September 21, 1973, reviewed various aspects of the design, construction, and performance of traditional sheeting systems and examined, in detail, recently developed technology for precast and cast-in-place diaphragm walls constructed using slurry trench techniques. It included recommendations for construction management and innovative contracting concepts for reducing construction costs and time.

Approximately 60 percent of the cost of a tunnel built by cut-and-cover methods is associated directly with the cost of the wall itself. The main advantages of diaphragm walls installed by the slurry trench method over conventional sheeting or soldier pile and lagging type construction are:

1. The same wall can be used as the temporary retaining wall and the permanent structural wall.
2. The movements associated with diaphragm wall construction and excavation inside the walls are small.
3. Diaphragm walls are watertight compared to conventional sheeting.
4. The construction operations are more compatible with the urban environment.

17. Key Words
tunneling, cut-and-cover tunneling, retaining walls, diaphragm walls, contracting practices

18. Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22151

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
UNCLASSIFIED	UNCLASSIFIED	190	

Form DOT F 1700.7 (8-69)

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WASHINGTON, D.C. 20590

LETTER OF TRANSMITTAL

The workshop, held in San Francisco on September 21, 1973, was intended for the benefit of designers and builders of tunnel sheeting and bracing systems. The workshop was jointly sponsored by the Federal Highway Administration and the Office of the Secretary, Department of Transportation, as part of their research and development efforts to improve transportation tunnel design and construction practices in the United States.

The proceedings are being sent to the participants of the workshop and to others known to be active in transportation tunnel work. Recipients of these proceedings who wish to receive similar future reports, developed within this research and development program, should write to Mr. F. J. Tamanini, Chief, Structures and Applied Mechanics Division, FHWA, HRS-11, Washington, D.C. 20590, indicating which aspects of tunneling are of particular interest.

Copies of these proceedings are also being sent directly to Federal Highway Administration Region and Division Offices. Sufficient copies are available to supply one copy to each Division Office and two to each State highway department. It is recommended that at least one of the State copies be forwarded to the department's bridge division.

Copies of the proceedings are available from the National Technical Information Service (NTIS) as noted on the title page. Information on the NTIS fee for supplying copies can be obtained from that agency.

Charles F. Scheffey
Charles F. Scheffey
Director of Research

FOREWORD

This document presents a written summary of lectures and notes presented at a workshop entitled "Cut-and-Cover" Tunneling at the U.S. Department of Transportation, First Federally Coordinated Program Progress Review sponsored by the FHWA Offices of Research and Development in San Francisco on September 21, 1973.

The workshop proceedings and notes are not intended to be a comprehensive treatise on the subject. The workshop was intended to give an overview of the various aspects of design, construction and performance of traditional sheeting systems with primary emphasis on recently developed technology for precast and cast-in-place diaphragm walls constructed using slurry trench techniques. The discussion includes consideration of the overall scope of application of these systems and their advantages and disadvantages in relation to each other and traditional construction methods. A presentation is made of selected aspects of design criteria and procedures, construction methods and the performance of actual installations. The presentation extends to construction management and innovative contracting concepts for reducing construction costs and time. For the most part, these notes follow the discussion as presented at the workshop. However, two papers not directly presented at the workshop have been added to the notes.

The individuals presenting the seminar are associated directly or indirectly with ECI-Soletanche, Inc., a specialty contractor in the field of diaghragm wall construction. As a natural result, most all of the construction equipment, examples, etc. are taken from the Soletanche experience. Moreover, considerable discussion is devoted to the subject of precast diaphragm wall, marketed under the name PANOSOL by Soletanche. The PANOSOL technique is patented by Soletanche, so all of the sites, equipment, methods, etc. involved in this technique are those of Soletanche.

TABLE OF CONTENTS

LETTER OF TRANSMITTAL i

FOREWORD ii

I. AN OVERVIEW OF SUPPORT SYSTEMS FOR CUT-AND-COVER TUNNELING WITH SPECIAL EMPHASIS ON DIAPHRAGM WALLS CONSTRUCTED USING THE SLURRY TRENCH METHOD 1

- METHODS FOR SUPPORTING EXCAVATIONS 2
- SOME COMMENTS ON SOLDIER PILES AND LAGGING AND STEEL SHEETING 6
- PARISIAN WALLS 8
- CAST-IN-PLACE CONCRETE DIAPHRAGM WALLS 10
- PRECAST DIAPHRAGM WALLS 16
- SUMMARY 20

II. ELEMENTS OF DIAPHRAGM WALL DESIGN 46

- WATERTIGHTNESS 50
- GROUND MOVEMENTS ASSOCIATED WITH EXCAVATION 52
  - Movements Related to Soil Type 53
  - Influence of Bracing and Wall Stiffness on Movements 55
- SUMMARY 61

III. CONTRACTING FOR UNDERGROUND SYSTEMS 85

- INTRODUCTION 85
- RISKS 86
- GEOTECHNICAL CONSIDERATIONS 91
- CONTRACTING BREAKTHROUGHS 92
- OVERRUNS 96
- CONTRACT MODEL 99
- ADVANTAGES AND DISADVANTAGES OF THE STANDARD LUMP SUM ADVERSARY APPROACH AND SUGGESTED DESIGN-CONSTRUCT INCENTIVE APPROACH 102

TABLE OF CONTENTS
(Continued)

- . CONTROLS REQUIRED 105
  - . Contingency Control 107
  - . Schedule Control 109
  - . Cost Control 110
  - . Engineering Control 112
- . SUMMARY AND CONCLUSIONS 114

IV. THE PREFABRICATED PANEL SLURRY WALL AND ITS APPLICATIONS 118

- . Introduction 118
- . Initial Applications of PANOSOL as Earth Retaining Walls 120
- . PANOSOL Applications Involving Composite Walls 121
- . PANOSOL Applications in Public Works 124
- . Watertightness of Slurry Walls 126
- . Outlook 128

V. GEOTECHNICAL PERSPECTIVES ON UNDERGROUND METROSYSTEMS 162

- . INTRODUCTION 163
- . GEOTECHNICAL ENGINEERING REQUIREMENTS 164
- . INTERRELATIONSHIP BETWEEN DESIGN AND CONSTRUCTION 167
- . THE PROBLEM OF MOVEMENTS 170
  - . Movements Causing Damage 171
  - . Movements Caused by Cut-and-Cover Construction 172
  - . Movements Caused by Tunnel Construction 176
- . SUMMARY 180
- . REFERENCES 181

I. AN OVERVIEW OF SUPPORT SYSTEMS FOR CUT-AND-COVER TUNNELING WITH SPECIAL EMPHASIS ON DIAPHRAGM WALLS CONSTRUCTED USING THE SLURRY TRENCH METHOD

This seminar deals with one aspect of the design of cut-and cover tunnels: the system for supporting the sides of the cut. The term support system is taken to mean both the temporary and permanent walls retaining the lateral earth pressures and the temporary bracing system for the walls which may involve internal struts or tie-back anchors. The primary emphasis of the discussion relates to techniques developed in Europe involving diaphragm walls constructed using the slurry trench method and to the relative advantages and disadvantages of diaphragm walls compared to traditional construction methods.

The question of the support system is one that should be studied comprehensively by planners, designers and builders of cut-and-cover construction. Approximately 60 percent of the cost of a tunnel constructed by the cut-and-cover method is associated directly with the cost of the wall itself. About 80 to 90 percent of the cost of building the tunnel structure is influenced very significantly by decisions about the type of wall employed.

The Department of Transportation has undertaken an ambitious research and development program whereby the DOT is seeking to reduce the cost of the federal tunneling program construction by 30 percent and to reduce construction time by a factor of 2 to 3. Because the single most important decision influencing both time and cost is the support system, engineers must look to improving the support system and the method of its construction in order to achieve the large sought after savings.

METHODS FOR SUPPORTING EXCAVATIONS

As listed in Table 1, five types of walls for supporting excavations are used extensively and typically. The first two of these methods provide only temporary support during the time when the earth is removed and the permanent wall is under construction. Both of these methods are well known and used extensively in the United States. The first method is the soldier pile and lagging type of wall known as a Berlin wall, because it was first used in Berlin in about 1890 in connection with open cut excavation. In common practice, the soldier piles are steel and the lagging is timber planks wedged between the piles as the excavation is advanced. The second method involves the use of steel sheet piles as the temporary wall support. Both these systems provide the temporary support for the excavation while the excavation is made; the permanent structure is constructed by forming a concrete wall inside the temporary excavation.

The remaining three methods listed in Table 1 combine both the function of the temporary support of excavation and the permanent retaining system into the same structure. One of these techniques is known in Europe as the "Parisian" wall; another is the cast-in-place concrete diaphragm wall; and the third is the precast concrete diaphragm wall. In each of these methods, the permanent wall is installed before making the excavation, and that wall then serves as the support during excavation before being finally incorporated into the permanent structure.

Other methods for supporting excavations have been used effectively. There are methods of freezing the ground along the excavation line, methods of pregrouting the soil prior to making the excavation,

METHODS FOR SUPPORTING EXCAVATION

Temporary Support Only

- Soldier Piles and Lagging
- Steel Sheet Piles

Temporary and Permanent Support

- Parisian Type Walls
- Cast-In-Place Concrete Diaphragm Walls
- Precast Concrete Diaphragm Walls

Table 1

as well as other techniques and variations. None of these methods are used very extensively, primarily because of their expense. Generally, such techniques are used to solve a specific problem that occurs due to localized unusual conditions or constraints.

Table 2 lists six factors that are important to consider in selecting the best type of wall for a particular solution. Environmental considerations are becoming increasingly important in urban environments. Such factors as disruption to the local businesses along the street, rerouting of traffic, noise and vibrations associated with the construction, dirt and dust on streets and walkways, and the total time that businesses and the community around the construction are inconvenienced are beginning to be regulated more and more by various agencies and thus, beginning to become important factors in design. In certain cases now, and probably even more so in the future, environmental considerations will dictate at least the general method of construction thereby eliminating certain other technical alternatives that are otherwise available to the designer. The various techniques for supporting an open cut excavation all have different impacts on the environment.

The second factor listed in Table 2 involves the requirements for both the temporary and permanent support. In addition to determining the loads acting on the wall, consideration should be given to whether or not the wall that supports the cut until the base slab and the roof of the structure are poured can also be incorporated into the permanent structure. Other important factors to consider, especially in a downtown area, involve movement of adjacent structures and utilities and dewatering. The various types of walls and different techniques of construction have a very significant

FACTORS TO CONSIDER IN WALL SELECTION

- Environmental Considerations
- Temporary and Permanent Support Requirements
- Movements of Adjacent Structures
- Temporary and Permanent Groundwater Control
- Surface Restoration
- Cost

Table 2

influence on both the movements of the adjacent ground and the problems of controlling the ground water. A fifth factor to consider deals with the surface restoration and the time the surface is disrupted. By incorporating diaphragm wall techniques into the construction plan it is possible to restore the surface very rapidly, before the major portion of the excavation and the structural construction is even started. Finally, there is the most important consideration - that of cost. Once a decision has been made between cut-and-cover and tunneling and once the grade and alignment of the road or subway has been chosen, the most important decision influencing overall construction cost involves the wall. As mentioned previously, about 80 to 90 percent of the money spent in getting the structure in the ground is very directly related to the type of wall that is chosen.

SOME COMMENTS ON SOLDIER PILES AND LAGGING AND STEEL SHEETING

Berlin walls, i.e. walls constructed using soldier piles and lagging, are generally the least expensive type of support system when only temporary support conditions need be considered. Berlin walls are particularly well adapted when factors such as ground water control and movement of adjacent buildings are not important considerations. The soldier pile and lagging system is very adaptable to different soil conditions, different geometries of excavation, etc.

The method involves placing a soldier pile, usually spaced on about 8 ft centers, slightly outside the line of the permanent structure. As the excavation proceeds, timber lagging, in some cases precast concrete lagging is inserted between the soldier piles. The technique is simple and the wall is relatively easy to construct with readily available construction equipment and trade skills.

Nevertheless, the method has a number of important disadvantages in many situations. The major technical problems with Berlin walls involve the need to underpin and the need to control ground water. The system is structurally very flexible. The lagging is installed as the excavation proceeds. Consequently, the wall is quite flexible below excavation level and is not very effective in preventing lateral inward movements of the soil below the base of the excavation. In most soils, the inward movement below excavation level as excavation proceeds is the most important component of the total lost ground. There are other problems associated with the lagging. The fastest and most economical method for placing the lagging is to wedge the timber just behind the flanges of the piles. However, in many soils it is not possible to trim the soil very well behind the lagging and consequently, voids remain behind the lagging and additional lost ground results. In most all cases, the lagging is left in place during the backfill process so that any void is eventually filled. A solution to this problem involves bolting the lagging to the inside of the soldier piles, enabling the soil to be trimmed accurately, but this increases cost.

Ground water cannot be controlled effectively with Berlin wall construction except by well point or deep well dewatering. There is no seal below the base of the excavation, therefore, if the dewatering is not properly accomplished, there is the danger of piping at the bottom of the excavation leading to lost ground. When dewatering is extensive, settlements are associated with the ground water lowering.

Steel sheeting eliminates several of the disadvantages of the Berlin wall. Ground water seal is established at the base of the excavation, provided the sheeting is driven in interlock, facilitating ground water control. The driven sheeting fits tightly to the soil. There is more resistance to inward movement below the base of the excavation because the sheeting does have considerable resistance to bending below the base of the cut. On the negative side, sheeting is usually slightly more expensive than the soldier pile and lagging system. Noise and vibrations associated with pile driving are an inconvenience in urban areas. Also, it is impossible in hard ground such as till or dense granular materials, or where there are obstructions.

PARISIAN WALLS

A Parisian wall is basically the same as a soldier pile and lagging wall in both design concept and sequence of construction. However, the wall serves both the function of a temporary support system and the permanent support system. The major difference from the Berlin wall is that the lagging, rather than being a temporary lagging, consists of poured in place concrete that becomes a permanent wall spanning between the soldier piles.

Two possible types of Parisian walls are shown in Figure 1. One involves using a precast concrete element with bent over starter bars placed in a predrilled hole and the other involves steel soldier piles.

Construction using the precast concrete elements involves the following steps. First a hole is drilled with an auger under a weak slurry grout having a compressive strength after setting of about 100 psi. The precast element is then placed in the hole and the grout allowed to set. Excavation is then made in lifts whose height depends on the character of the ground. The excavation is made flush with the outside face of the soldier elements as shown in Figure 1. Once the elements are exposed, the starter bars are bent out and reinforcing steel is placed for the panel. Forms are placed at the inside face of the wall and the wall is concreted or, alternately, shotcrete can be used. Then the next lift is excavated. In this way, the wall is constructed from the top down. Note that the horizontal reinforcing steel in the wall is, in the end product, continuous through the soldier element.

A variation on this technique is shown on the lower portion of Figure 1 and involves placing steel soldier piles, either in predrilled holes or by driving. A mesh or light cage is placed between the soldier piles as the excavation proceeds in lifts and then shotcrete is applied. For excavations having depths of 30 to 40 feet the soldier pile spacing would be about 4 to 6 ft and the shotcrete thickness about 8 inches. When used as a permanent wall, this type of construction is considerably less expensive than classical construction using Berlin walls and forming concrete.

Parisian walls, of course, have the same disadvantages and suffer from the same difficulties as Berlin walls. Movement of adjacent ground is not well controlled and there is no way of controlling the ground water other than by dewatering. In fact, it is very difficult to use Parisian walls unless the ground water is lowered below the base of the cut, because of difficulties in concreting with seepage from the face.

Figures 2 through 4 show photos of a wall in Venezuela constructed using soldier piles and shotcrete. The excavation is for three basement levels of a large development in Caraças and is 40 to 45 ft deep in cohesive granular alluvial deposits. The initial ground water level is approximately 15 ft below ground surface and was lowered during construction by deep wells. The basement was provided with a permanent dewatering system involving underdrains and sumps, so for the permanent condition the head acting on the wall is only about 15 ft. Approximately 150,000 ft^2 of permanent basement wall were constructed using this technique at very low cost with very good performance.

Figure 2 shows a view of the partially completed wall. Permanent tie-backs were used for support because the basement floors of the structure placed in the excavation are not continuous. The soldier piles were driven rather than placed in predrilled holes and a number of piles are out of alignment. This presents no particular problem as long as the piling remains in the plane of the wall. The close up view of the excavation shows ponded water which is flowing from perched levels. Figure 3 shows the excavation after cleaning and trimming the face of the cut. Note that the vertical reinforcing steel will be lapped. Figure 4 shows the steel in place and several panels partially concreted.

At various stages in the construction of this site, four different types of walls were installed: cast-in-place diaphragm walls, conventional Berlin walls with the formed concrete inside, steel sheetpiling with formed concrete inside, and the Parisian wall described above. The Parisian wall was one-half the cost of the least expensive of the other three. The performance of all four wall types in terms of movement, ground water control, etc. was comparable.

CAST-IN-PLACE CONCRETE DIAPHRAGM WALLS

The technique for installing the classical cast-in-place concrete diaphragm wall or slurry wall is shown schematically in Figure 5. The wall is installed from the ground surface before general excavation is begun. Construction involves three steps: excavating a panel, placing the reinforcing cage and concreting. The panel is excavated using one of a number of specialized types of trenching equipment. During excavation the trench is kept full with a bentonite and water slurry which supports the sides of the trench and prevents collapse during excavation and prior to concreting the panel. The panels are classified as primary panels and secondary panels -- primary panels all excavated and concreted first leaving the secondary panels (the space between) to be completed later ultimately forming a continuous wall with joints between the adjacent primary and secondary panels. Once trench excavation for a panel is complete, the subsequent operations are to position a reinforcing steel cage in the slurry filled trench and finally to replace the slurry by tremie concrete. The conventional procedure for joining primary and secondary panels is illustrated in Figure 5. Before the concrete is tremied, a pipe is placed at the ends of each primary panel. The concrete is then tremied and as the concrete begins to set, the pipe is moved slightly to break the bond between the steel and concrete. Once the concrete is set sufficiently to stand, the pipe is withdrawn completely so that the ends of the primary panels are concave. The excavating tools used for the secondary panels fit the shape of the end of the primary panel, cleaning and scratching the joint. Once the secondary panel trench is completed, the cage is positioned and the panel poured. In this way, a partially keyed cold joint is formed.

The main advantages of diaphragm walls as compared to conventional construction are that the wall is both the temporary and permanent support, the ground movements are small and the ground water seepage into the excavation is controlled or cut-off. The movements associated with diaphragm wall construction and excavation inside of a diaphragm wall are very small compared to any other construction technique. There is no need to underpin or otherwise protect adjacent structures. In fact, in many situations, diaphragm walls are used in lieu of underpinning. Properly constructed diaphragm walls are watertight so they can be used to control the ground water level, both for temporary conditions and also for the permanent condition. There are other advantages including a significant reduction in construction time and disruption of street, and the construction process is relatively quiet and vibration free.

Usually, the first step in constructing a wall is to build a pretrench, a concrete wall about three feet deep which serves two purposes: to guide the excavating equipment and to prevent the top of the trench from caving due to the up-and-down movement of the surface of the bentonite slurry. A pretrench under construction is shown in Figure 6.

An essential feature of diaphragm wall construction is the bentonite water slurry that supports the wall of the panel excavation during trenching. Figure 7 shows the vortex-type mixer commonly used to prepare the slurry. A known volume of water is introduced in the mixer and then circulated at high velocity as the required amount of bentonite is added. The mixing is continued until the bentonite is properly hydrated. The property of the slurry that is indicative to the qualities required for diaphragm wall construction is viscosity. Viscosity is measured on the site using a Marsh Cone Test where standard volume of bentonite is allowed to flow from a standard funnel and the time required is measured. The length of time in seconds is the Marsh Cone viscosity. The quantity of bentonite per mix required to yield a given viscosity will vary by a factor of as much as two or three depending on the grade of bentonite used.

A wide variety of trenching equipment has been developed for diaphragm wall construction. At first the most common tool was the mechanical grab where the clamshell is hung free from one cable and closed by a second cable. Various companies have developed and improved the mechanical grab to give more closing power. The most common method is to run hydraulic lines down to the clamshell, operating powerful rams to close and open the bucket. ECI-Soletanche usually uses a hydraulic powered bucket rigidly attached to a vertical Kelly-bar such as shown in Figure 8

Obstructions such as boulders, concrete foundations, old wood etc., present the major problem in trench excavation. Anything larger than the width of the panel cannot be removed from the panel with the grab alone. Chisels or star-drills are used to break up obstructions to a size that permits their removal. Figure 9 shows a chisel weighing about three tons. Chisel configurations vary according to the nature of the material to be broken; different designs are more effective for hard rock, soft rock, wood, etc. Obstructions reduce the rate of excavation and increase cost. In addition, obstructions, or even large gravel, prevent the excavating tools from neatly trimming the sides of the trench. Because the ground is the form for the concrete, the exposed wall may be rough with bulges due to the overbreak. When excavating in clay and relatively uniform fine sand or cohesive sand, the surface of the completed wall is surprisingly smooth. Coarse gravel and loose fill cause the wall to be rougher. Usually, the roughness is less than 2 to 3 inches in these materials except for localizing bulges that are easily broken off during excavation.

Another general method for excavating panels involves the use of a reverse circulation drill, mounted on a carriage that runs on rails. The panels are formed by drilling a hole at each end of the panel and then moving the chopping bit back and forth along the panel; each pass removes about one foot of depth. The slurry is pumped by reverse circulation up through the drill tools, through a desanding unit and back into the trench. This equipment, shown in Figure 10, is still the most efficient method to key into rock or to drill through extermely hard soil, but is used relatively little for other applications because of the greater efficiency of hydraulic clamshells.

Once the panel has been excavated, the mud in the panel must be desanded to remove the cuttings suspended in the slurry. If sand and gravel in the slurry are not removed, problems with the concreting leading to poor concrete or sand pockets in the concrete are almost inevitable. Desanding is usually accomplished with a unit similar to that shown in Figure 11. It consists of a vibrating screen to remove larger particles and cyclone-type centrifigual separator to remove the smaller size suspended particles. After excavation is complete, the slurry in the trench is circulated through the desander and clean slurry is returned to the trench. Placing the cage and concreting proceeds immediately thereafter.

After the panel is complete and the slurry desanded, a "cage" of reinforcing bars is lowered into the trench. Spacers are used to guide the cage and to hold it in position in the panel. Figure 12 shows a cage being lifted into position for placing in the trench. The cage may contain reservations for installing tie-backs during the general excavation, a built-in wale made by placing extra horizontal steel at the support elevation, reservations for utilities, keys for floor slabs or bent over starter bars.

Figure 13 shows a tremie concrete operation. When a panel is longer than about ten feet, two tremie pipes should be used. The tremie pipe, typically an eight inch flush joint heavy duty pipe, extends to a depth just above the bottom of the panel. As the level of the tremie concrete rises displacing the slurry, the pipe may be shortened, but the bottom of the pipe is always kept at least ten feet below the surface of the concrete. The bentonite slurry displaced by the concrete is pumped into a storage tank after passing through the desander once again and is reused in subsequent panels. The tremie concrete is a special mix, usually having a slump of about eight inches, a high sand to stone ratio, small stone size and a retarder additive.

Figure 14 shows a view of a job site where several types of equipment are being used. The Kelly mounted grabs are excavating in soil and the reverse circulation drills are keying the bottom of the panels into bedrock. The bentonite mixing station and the desanding unit are positioned to the left of the site entrance near the top of the photograph. Stop-end joint tubes are on the ground at the lower left corner of the picture.

One of the chief construction problems and a major source of flaws in the finished wall is the joint between panels. A method which has been widely used is the joint-tube or stop-end tube, a pipe whose diameter is slightly less than the width of the trench placed at both ends of the primary panel. As described previously, the pipes are removed slowly using hydraulic jacks after the concrete has begun to set. The resulting hole has the shape of the clam shell and forms the ends of the secondary panels.

Various methods have been used to improve on the watertightness and ease of construction of the joints between panels. Figure 15 shows several examples. One method to seal the joints is grouting. A grout pipe is placed in the joint and pressure grouted after the two panels are completed. A keyed joint can be constructed by attaching a small pipe in the stop-end tube used for the primary panel concreting. A double-key joint and even a water-stop joint can be constructed using this procedure as shown in Figure 15.

Because there are a number of construction problems caused by joint tubes, particularly the overtime working hours and the possibility of having a joint-tube stuck in the concrete, several other types of joints have been developed. Two of these are shown in Figure 15. The first one, to be used with round-ended clam shell buckets, is a precast concrete beam that is lowered into a small primary panel and set into place with a weak grout. The main panels are then excavated as secondary panels. The second method uses steel joint beams put at both ends of primary panels. The outer spaces between the flanges are filled with styrofoam held in place by wire mesh. The styrofoam is ripped out during the excavation of the secondary panels with square ended grabs. This type of joint is easy to construct and results in a better sealed joint with more structural continuity than the ordinary stop-end joint. A number of other variations have been used, including joints that allow continuity of the horizontal reinforcement.

PRECAST DIAPHRAGM WALLS

Figures 16 and 17 show schematic views and photographs of two types of precast diaphragm walls. The procedure for installation of precast diaphragm walls is somewhat similar to that of cast-in-place diaphragm walls except that construction is faster, involving fewer operations. First a pretrench is constructed. Then the trench is excavated using the same equipment as for a conventional slurry wall. Trenching is carried out continuously; normally there are no primary or secondary panels. The slurry supporting the sides of the trench is a special bentonite-cement mix, a type of grout called "coulis" (from the French term for liquid grout), having additives to control its viscosity, stiffness, setting time and strength. Typically, the coulis is controlled so that it sets after two or three days and achieves a strength between approximately 100 and 200 psi. The coulis serves to maintain the stability of the trench during excavation and while placing the precast panels. After setting, the coulis seals the panels to the ground and to each other. The panels are inserted in the trench one after another as the trenching proceeds forming a continuous wall. The panels are constructed of high quality concrete and are positioned accurately in the trench and suspended from the pretrench until the coulis hardens. An anti-adhesive coating is used on the inner face of the panels so that the hardened coulis on the exposed face of the wall can be easily stripped away during excavation. The grout on the soil side of the wall remains in place as does the grout in the joints between panels. The hardened grout is plastic and serves as a watertight barrier. This type of diaphragm wall is known as a PANOSOL wall.

The PANOSOL wall shown in Figure 16 is composed of interlocking tongue-in-groove panels. Note the smooth appearance of the wall and how well the key for the base slab lines up between the adjacent panels. One of the great advantages of this process is that the wall can be positioned very accurately -- within about one-half inch. The wall is absolutely vertical because it is plumbed by gravity when inserted in the trench.

Many different panel geometries can be used depending on the structural action required. Figure 17 shows a wall composed of T-beams and slabs in what amounts to a soldier pile and lagging configuration. The T-beam extends considerably below the base of the excavation to develop passive resistance and the thin concrete slabs act as lagging bearing on the T-beams. The slabs extended just below the base of the excavation to prevent localized stability problems.

A frequent application of the PANOSOL wall is as composite wall, i.e. the structure of the wall varies over its height in accordance with its function. An example is shown in Figure 18. The upper portion of the wall is a retaining wall and the lower portion a water cut-off requiring no structural strength. The soil below the basement slab was to be permanently dewatered so that the slab was not designed for up-lift and was not waterproof. The resulting design called for a coulis filled trench socketed into an impermeable layer with prefabricated concrete panels placed in the upper portion. In this manner the upper portion serves as a retaining wall for the excavation and the lower portion serves as a low cost water cut-off.

Figure 19 shows a precasting site, with the elements stacked in storage. All factors considered, PANOSOL walls cost about the same or slightly less than cast-in-place diaphragm walls, provided that the precasting can be done on site. The mixing plant for coulis operation is more complicated because there are additional components added to the basic bentonite-water mix. The first PANOSOL project in the United States, a short length of sea-wall, is seen in Figure 20. Because the wall was shallow, about 25 ft deep, trenching was carried out with a backhoe rather than clamshells. Figure 21 shows a PANOSOL site in Paris where precast panels are being installed in a narrow street for a subway line.

Figure 22 shows a wall where plates have been precast into the panel so that starter beams for floors could be welded. Dowel bars, starter bars, shear keys, reservations, etc., can also be placed in the precast concrete and positioned accurately. Moreover, the reinforcing bars in the concrete can be positioned accurately and good quality vibrated and steam cured concrete can be used. As a result, a thinner wall is possible. Also, additives can be used in the concrete to improve watertightness and the soil side of the panel can be coated with bitumen or other waterproofing agents. The central panel in the Figure 22 photograph is a test panel. The surface of this panel was provided with an architectural finish of mosaic tile before inserting the panel in the trench. The tile was coated, the panel inserted in the trench and the grout scraped away.

A new method for constructing cut-and-cover tunels under city streets in part, prompted development of the PANOSOL process. Two recent applications of the process are illustrated schematiclly in Figure 23. The process is known as the "under-the-roof" method and involves constructing the permanent walls of the tunnel from street level, excavating to the roof level, pouring the roof against the ground, and backfilling over the roof while excavating under it. The base slab and interior work are completed under cover. This method is not limited to PANOSOL type walls although PANOSOL walls are better adapted because of the speed of construction. Figure 24 shows a schematic process as it was used on the A13 auto-route extension in Paris. The diaphragm walls were bearing walls in this case and, to support the heavy roof loads due to the long span, the lower portion of the panels were filled with tremie concrete and the precast panels implanted in the tremie concrete while fresh. The job site shown in Figure 21 employed a similar procedure for a double box subway.

The great advantage of the under-the-roof method as compared to conventional construction is that disruption of the surface is minimized. The street is disrupted only once rather than twice as in conventional construction where temporary decking is used. The time of disruption is also reduced. In most cases the slurry wall can be installed, excavation completed to roof level, the roof concreted, the excavation backfilled and the street restored in roughly the same time as required to install soldier piles and install temporary decking with conventional construction. Also, the need for temporary decking is eliminated, resulting in a saving of 5 to 10% of the total structural cost. Noise and vibration are also reduced as are temporary bracing requirements because the roof slab replaces at least one level of temporary bracing. As environmental constraints become more stringent, the under-the-roof method will undoubtedly be used more and more in urban underground construction.

SUMMARY

Recent studies analyzing the cost of cut-and cover subway construction have shown the following typical breakdown of the total structural cost:

Construction Item	Percentage of Total Cost
Excavation and permanent structure	30-40%
Temporary retaining wall	30%
Utility maintenance and relocation	5-10%
Underpinning	5-10%
Decking	5-10%
Other (backfill, restoration, mobilization)	10%

It is obvious from this table that the major part of the structural cost is directly related to the method chosen for supporting the excavation. Certainly, support system considerations should dominate any economic analysis and design of cut-and-cover construction. When selecting the type of wall and method of construction, factors influencing design such as those indicated in Table 2 should not be isolated, the overall system and cost should be evaluated.

The principal advantages of diaphragm walls installed by the slurry trench method over conventional sheeting or soldier pile and lagging type construction are:

1) The same wall that is used as the temporary retaining wall can be the permanent structural wall.

2) The movements associated with diaphragm wall construction and excavation inside a diaphragm wall are small compared to other

construction techniques. Underpinning or other special protection of adjacent structures is not required.

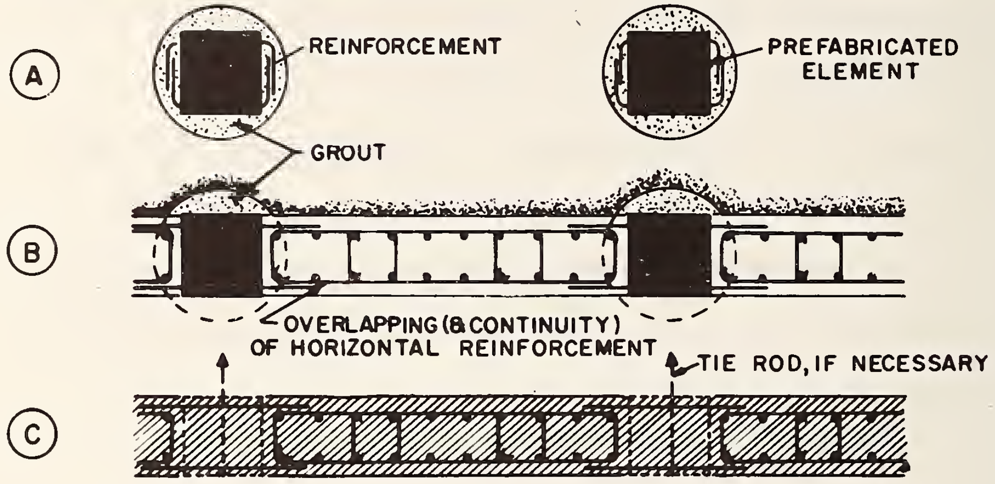
3) Diaphragm walls are watertight compared to conventional temporary sheeting, thereby reducing dewatering costs, increasing safety and protecting adjacent structures from settlement.

4) The method of construction is more compatible with urban environments. There is no pile driving causing noise. If the "under-the-roof" method of construction is used, the savings in time, and decrease in disrutpion and noise are further improved.

PARISIAN WALLS

WALLS BUILT FROM TOP-DOWN AS EXCAVATION PROCEEDS

I PRECAST SOLDIER ELEMENT



II STEEL SOLDIER PILES AND SHOTCRETE

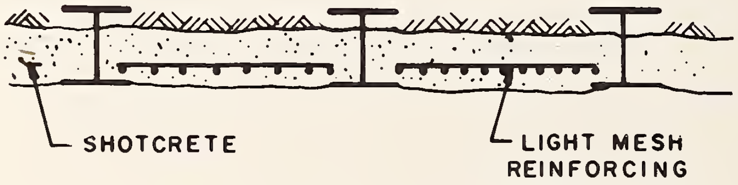
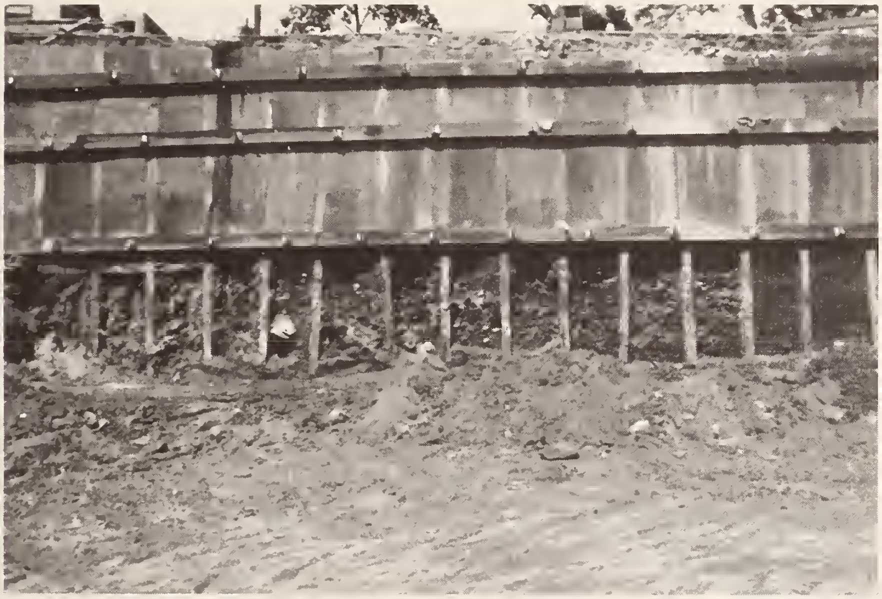
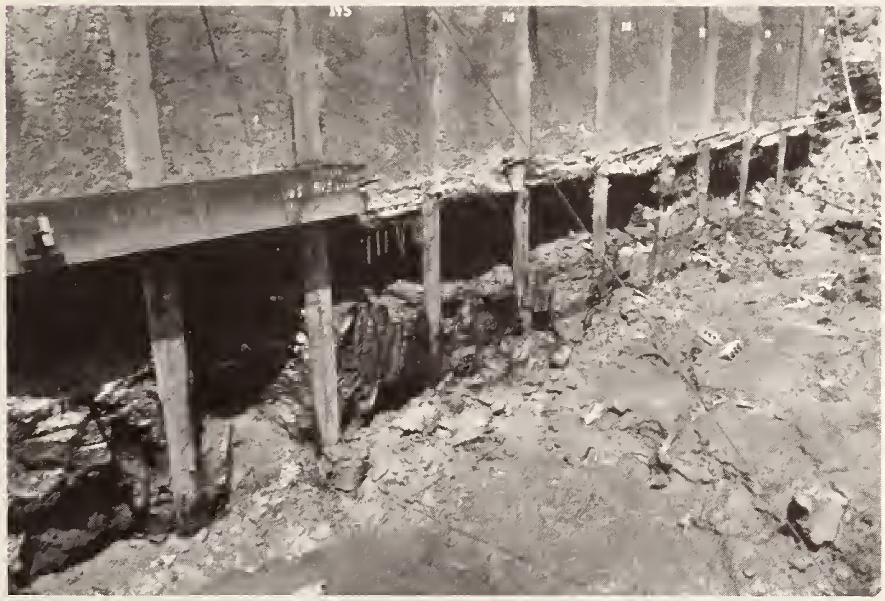


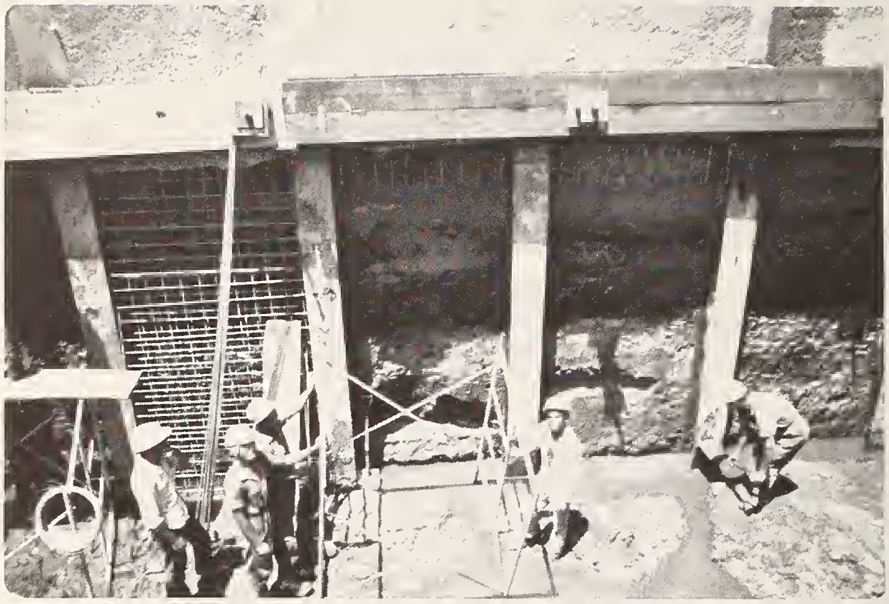
FIGURE I





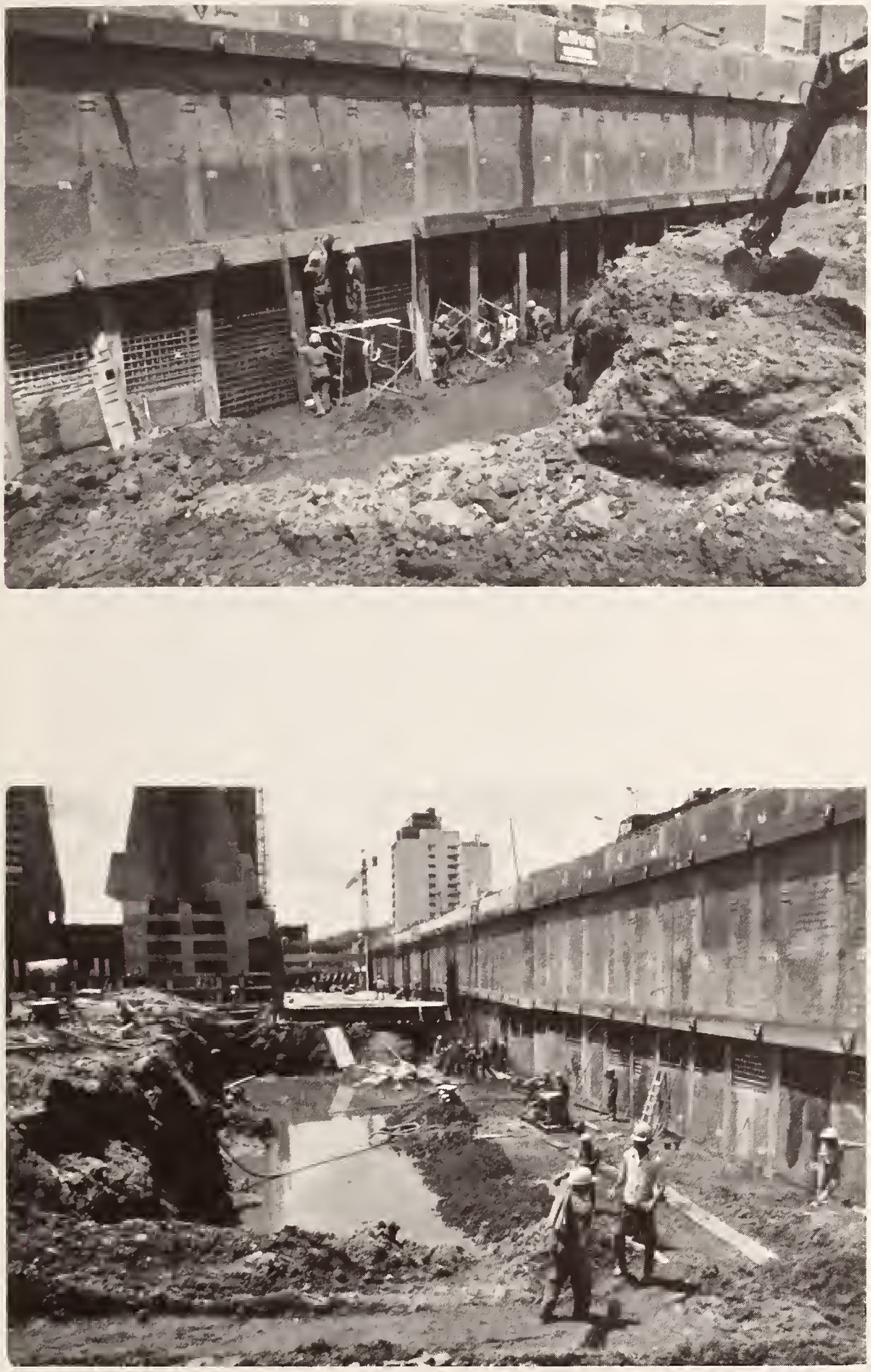
VIEW OF PARTIALLY COMPLETED PARISIAN WALL CONSTRUCTED OF SOLDIER PILES AND SHOTCRETE

FIGURE 2



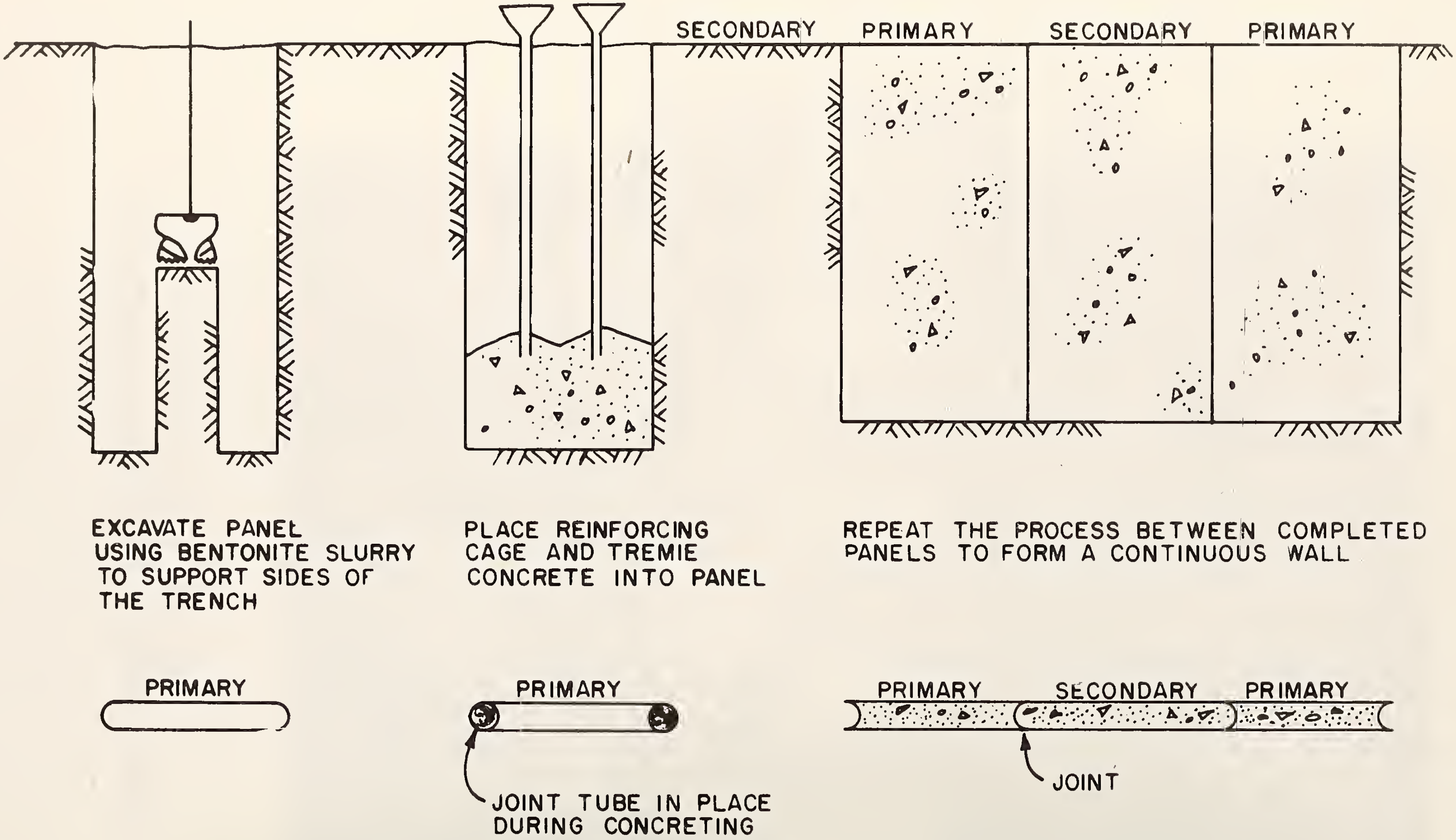
VIEW OF EXCAVATION LIFT FOR PARISIAN WALL AFTER TRIMMING SOIL AND PLACING REINFORCING STEEL

FIGURE 3



VIEW OF PARISIAN WALL
DURING VARIOUS STAGES OF CONCRETING

FIGURE 4



EXCAVATE PANEL USING BENTONITE SLURRY TO SUPPORT SIDES OF THE TRENCH

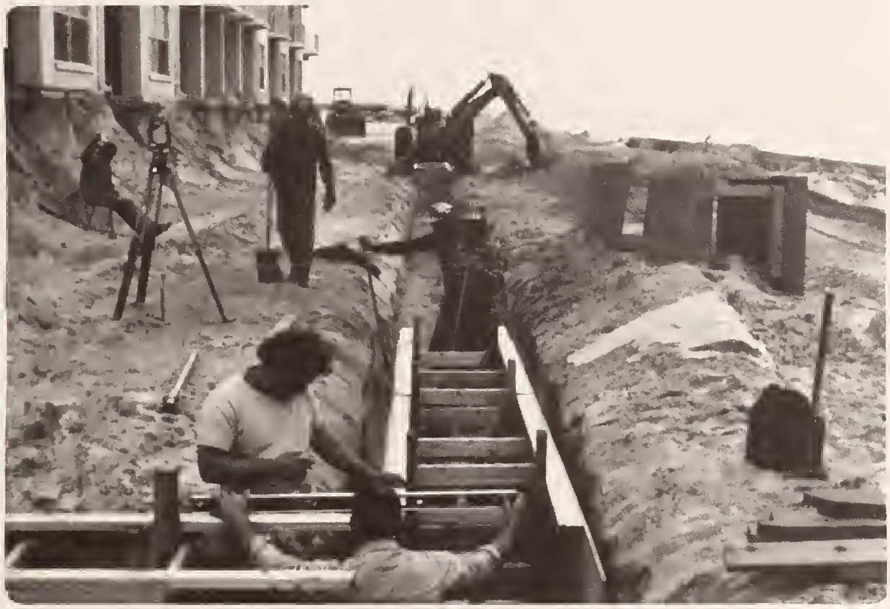
PLACE REINFORCING CAGE AND TREMIE CONCRETE INTO PANEL

REPEAT THE PROCESS BETWEEN COMPLETED PANELS TO FORM A CONTINUOUS WALL

JOINT TUBE IN PLACE DURING CONCRETING

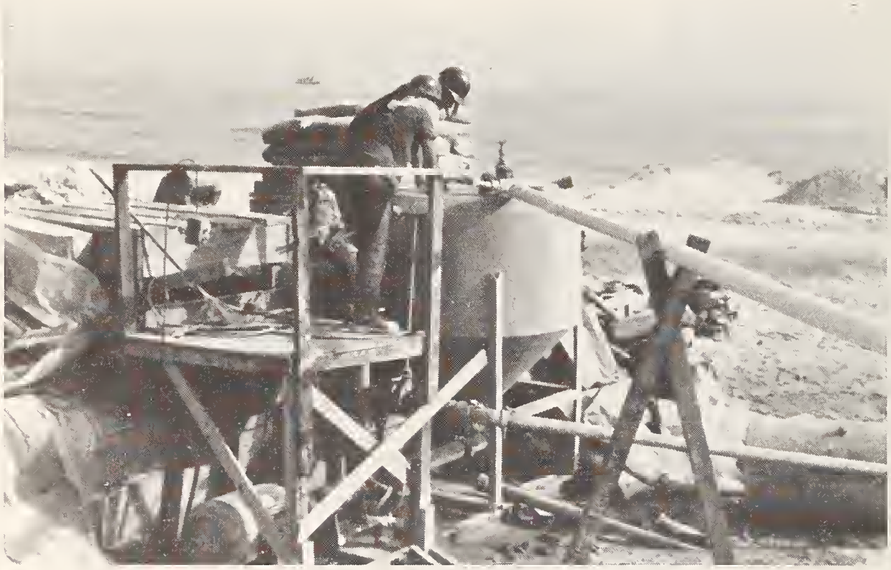
SCHEMATIC OF METHOD FOR CONSTRUCTING COST-IN-PLACE CONCRETE DIAPHRAM WALLS

FIGURE 5



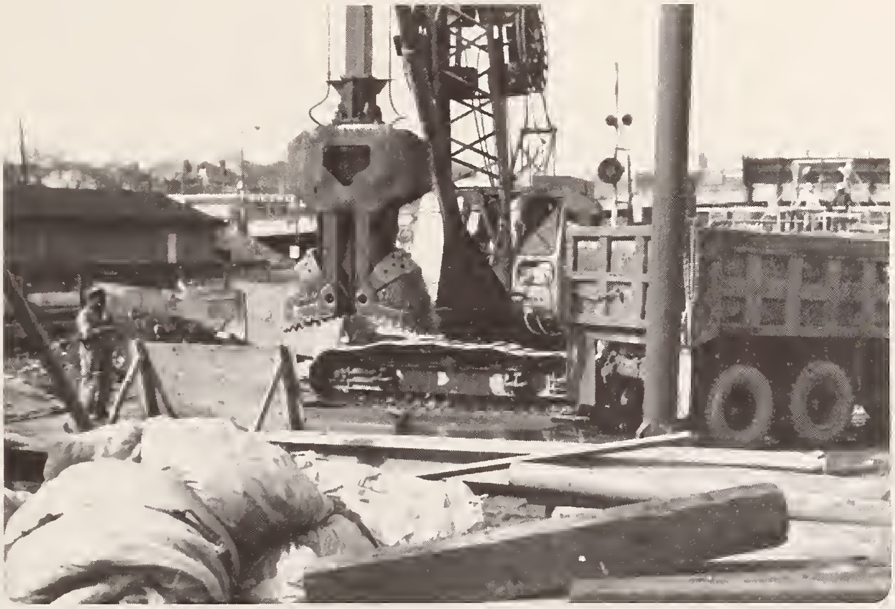
PRETRENCH UNDER CONSTRUCTION

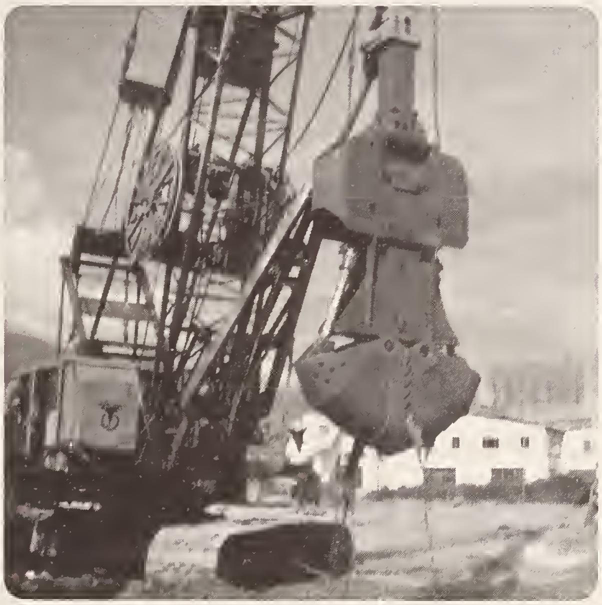
FIGURE 6



HIGH VELOCITY VORTEX MIXER
USED TO HYDRATE BENTONITE SLURRY

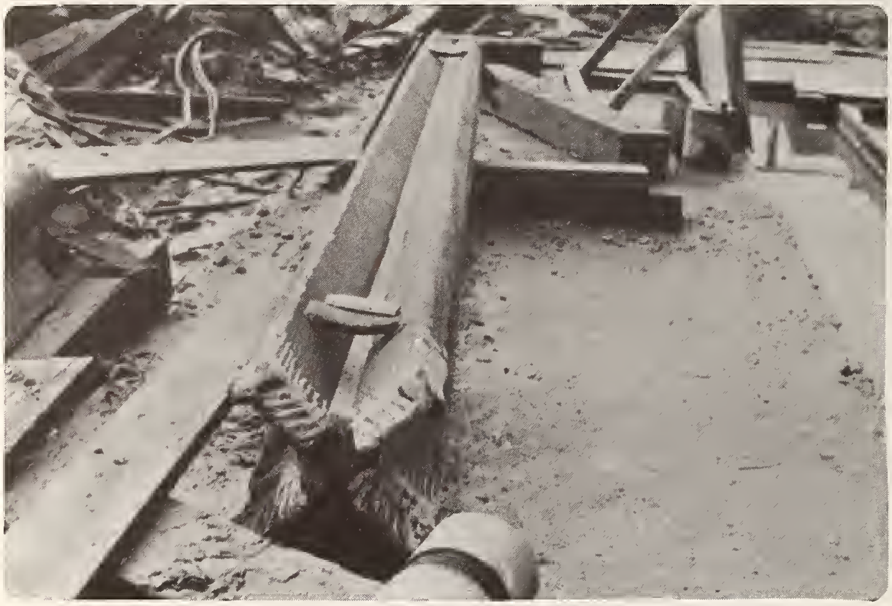
FIGURE 7





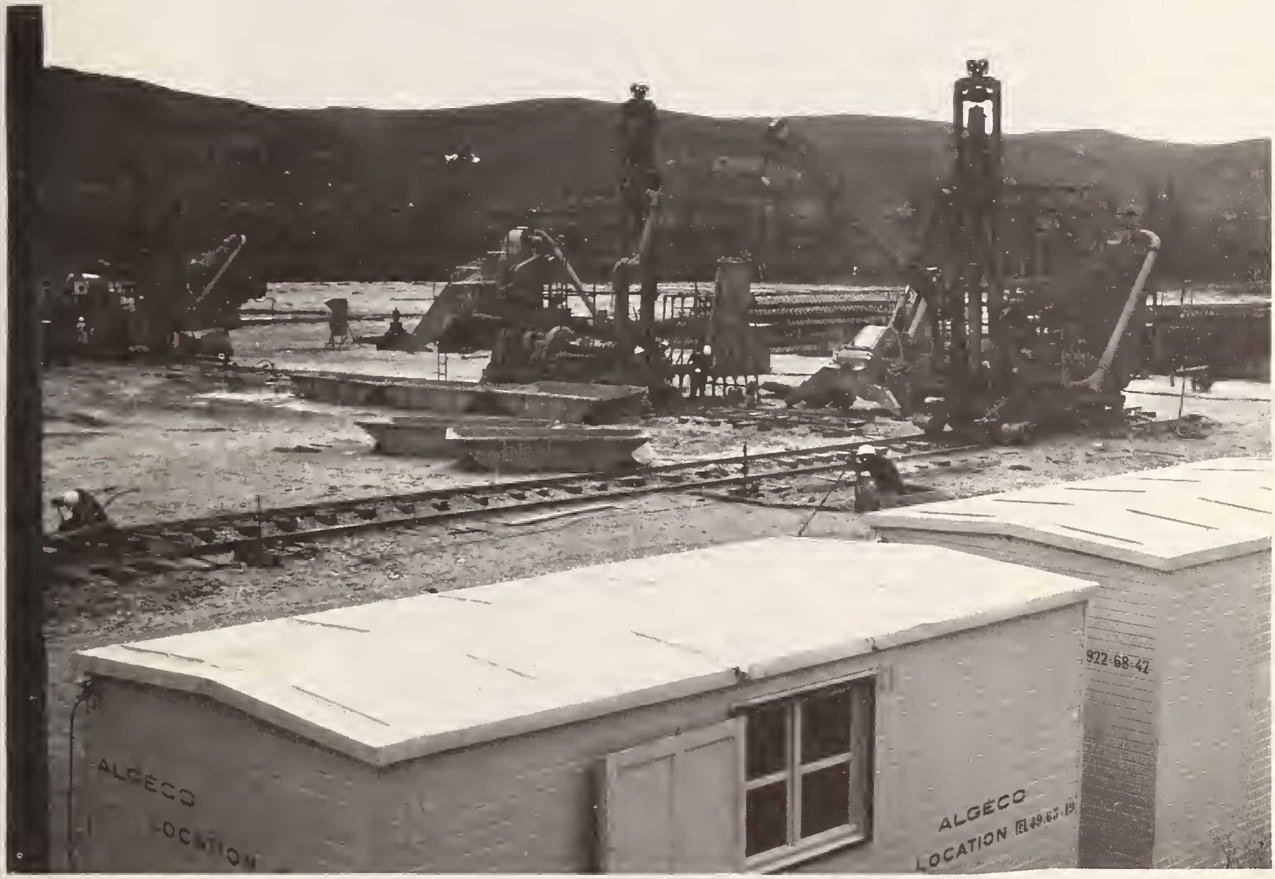
HYDRAULIC CLAMSHELLS MOUNTED
ON KELLY BAR

FIGURE 8



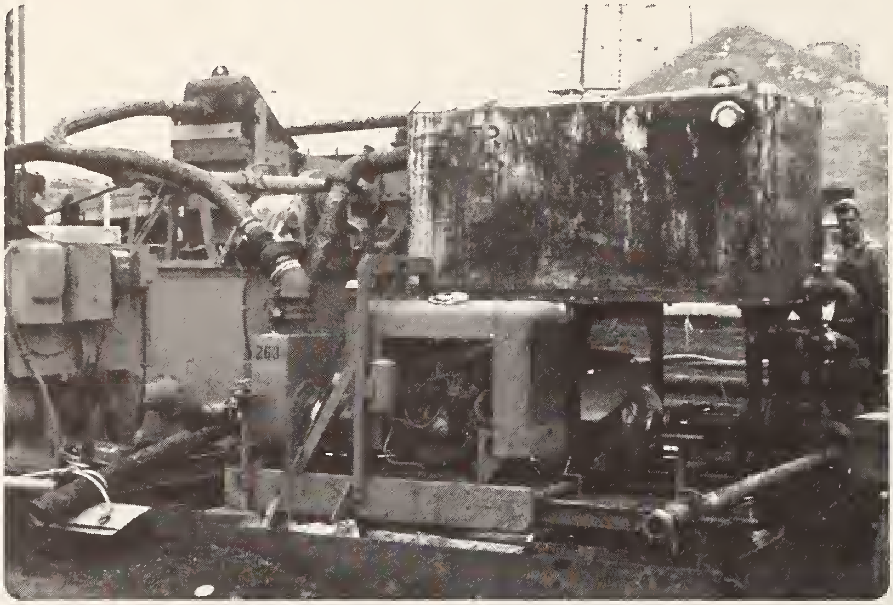
CHISEL USED FOR BREAKING
HARD ROCK OR CONCRETE OBSTRUCTION

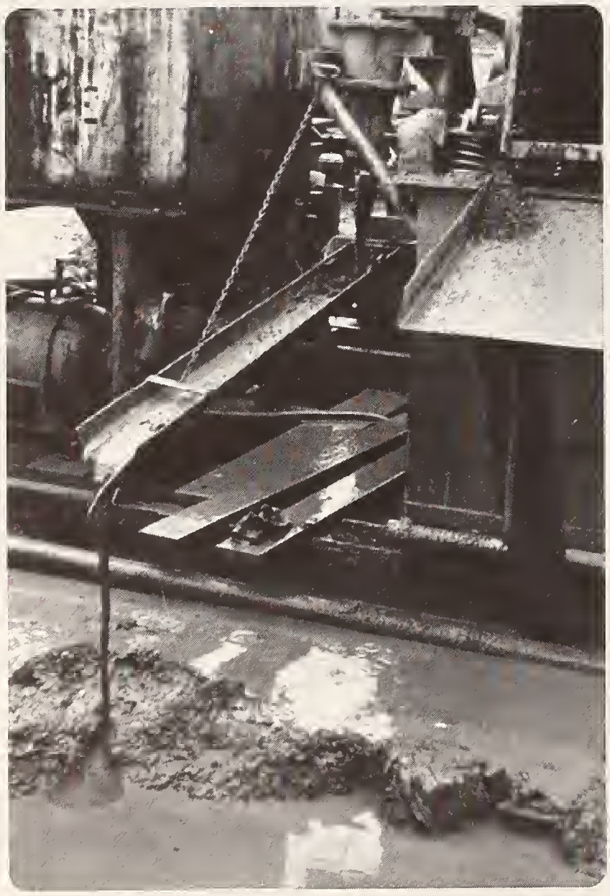
FIGURE 9



SLURRY WALLS BEING CONSTRUCTED USING
REVERSE CIRCULATION DRILLING EQUIPMENT
FOR EXCAVATING PANELS

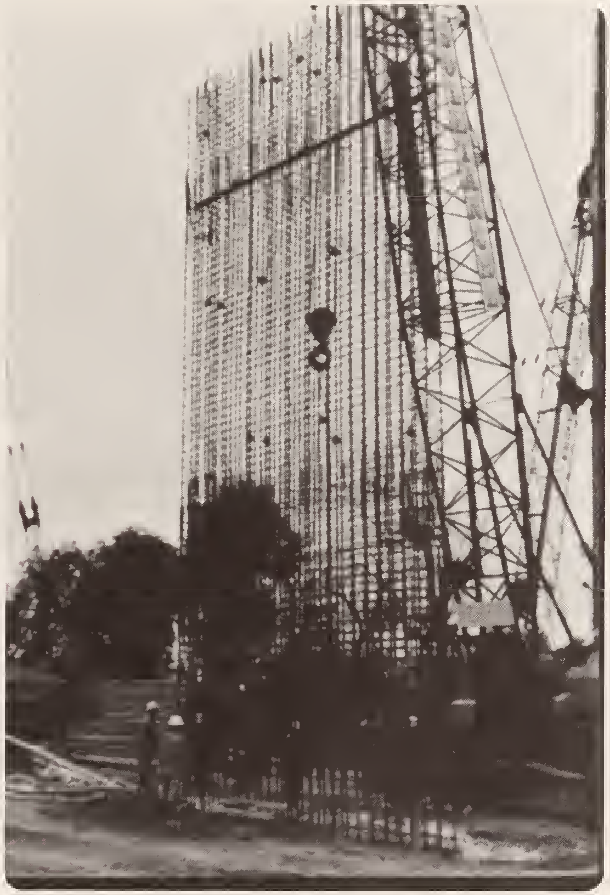
FIGURE 10





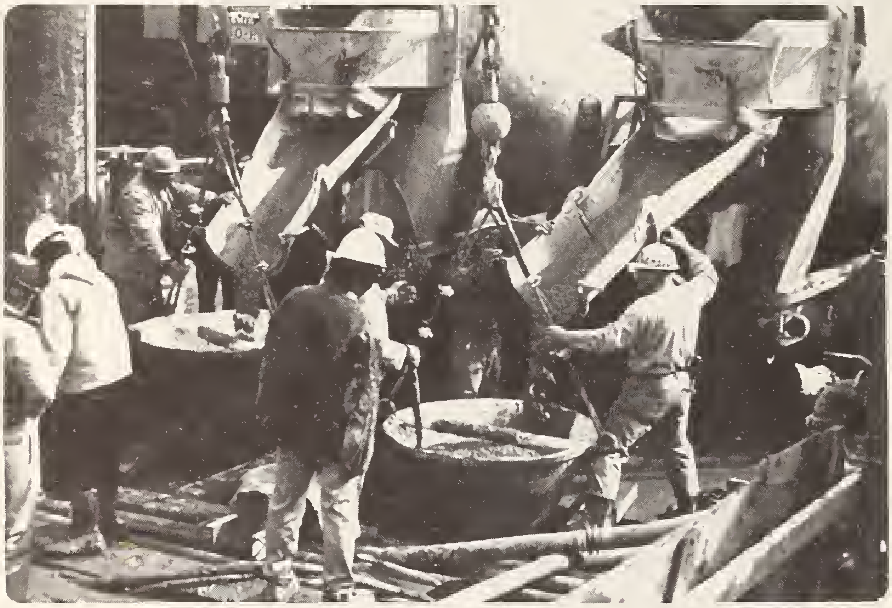
DESANDING UNIT USED TO REMOVE CUTTINGS FROM BENTONITE SLURRY

FIGURE 11



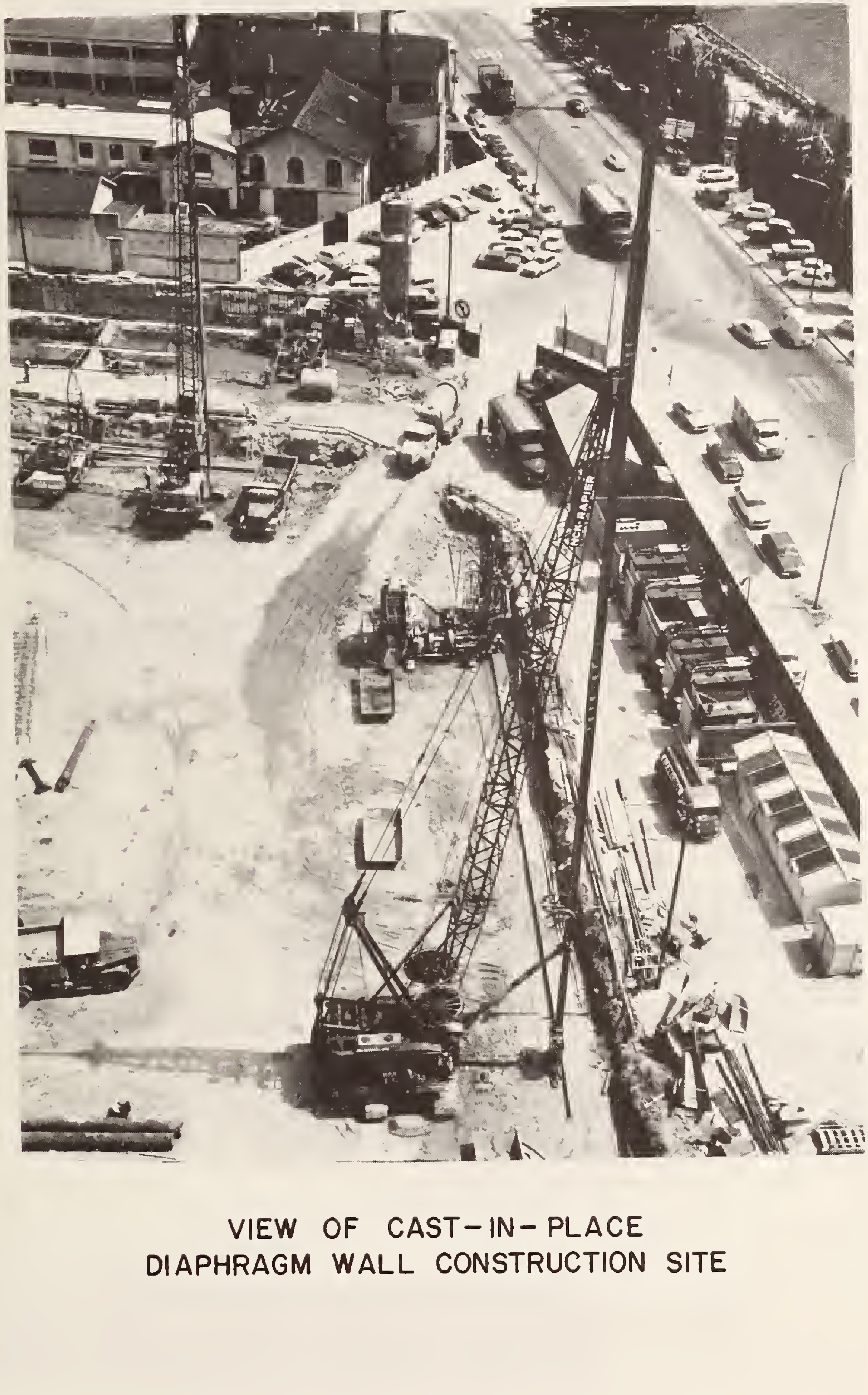
REINFORCING CAGE
BEING LOWERED INTO TRENCH

FIGURE 12



TREMIE CONCRETE BEING POURED
FOR DIAPHRAGM WALL PANEL

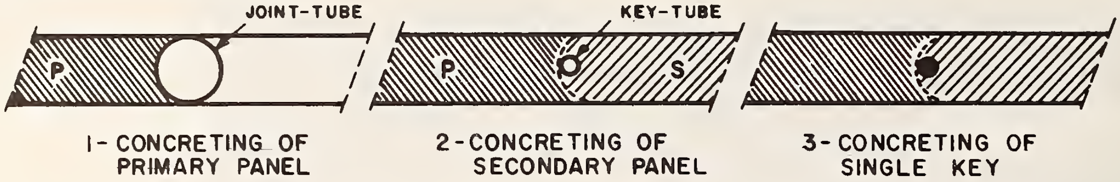
FIGURE 13



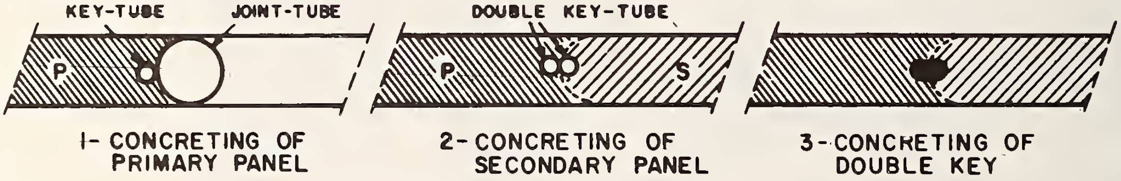
VIEW OF CAST-IN-PLACE
DIAPHRAGM WALL CONSTRUCTION SITE

FIGURE 14

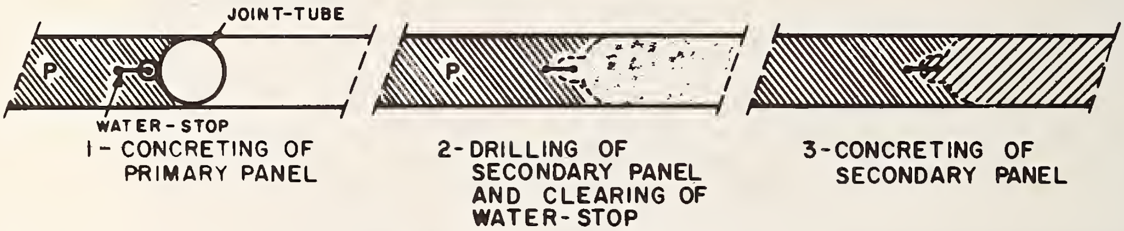
SINGLE KEY JOINT


JOINT-TUBE
KEY-TUBE
P
P
S
1-CONCRETING OF PRIMARY PANEL
2-CONCRETING OF SECONDARY PANEL
3-CONCRETING OF SINGLE KEY

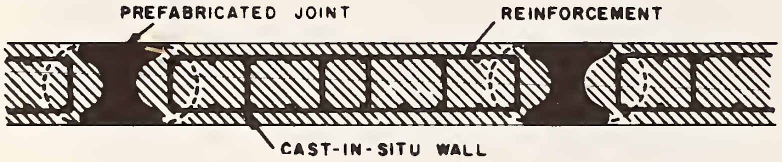
DOUBLE KEY JOINT


KEY-TUBE
JOINT-TUBE
DOUBLE KEY-TUBE
P
P
S
1-CONCRETING OF PRIMARY PANEL
2-CONCRETING OF SECONDARY PANEL
3-CONCRETING OF DOUBLE KEY

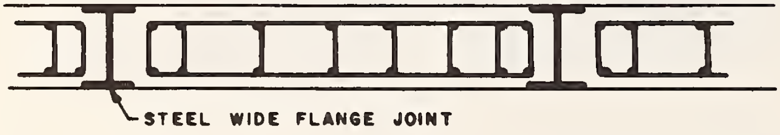
WATER-STOP JOINT


JOINT-TUBE
P
WATER-STOP
P
1-CONCRETING OF PRIMARY PANEL
2-DRILLING OF SECONDARY PANEL AND CLEARING OF WATER-STOP
3-CONCRETING OF SECONDARY PANEL

PREFABRICATED JOINT

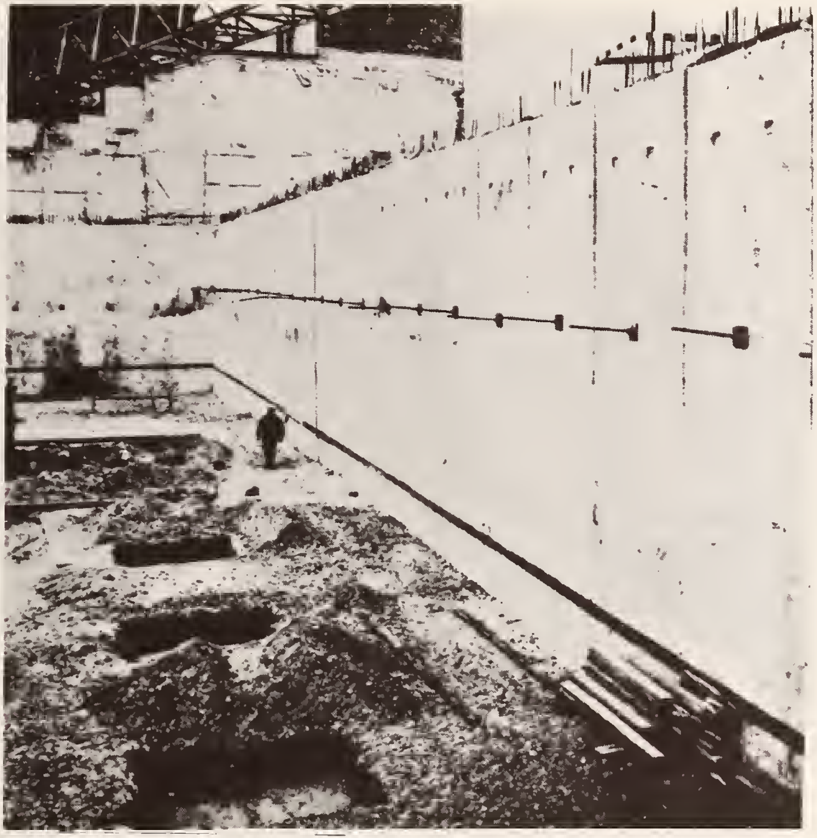


STEEL BEAM JOINT



VARIOUS TYPES OF JOINTS FOR CAST-IN-PLACE DIAPHRAGM WALLS

FIGURE 15



TONGUE - IN - GROOVE PANELS

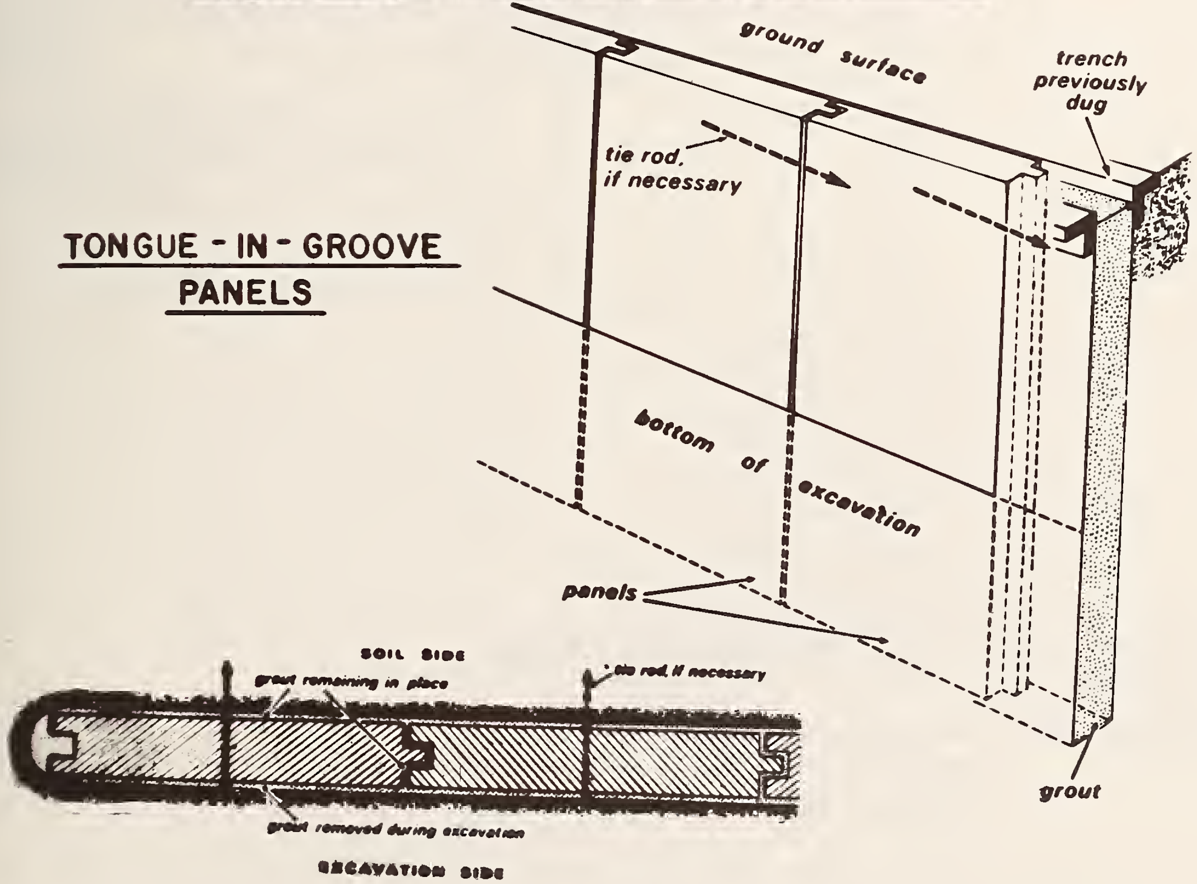
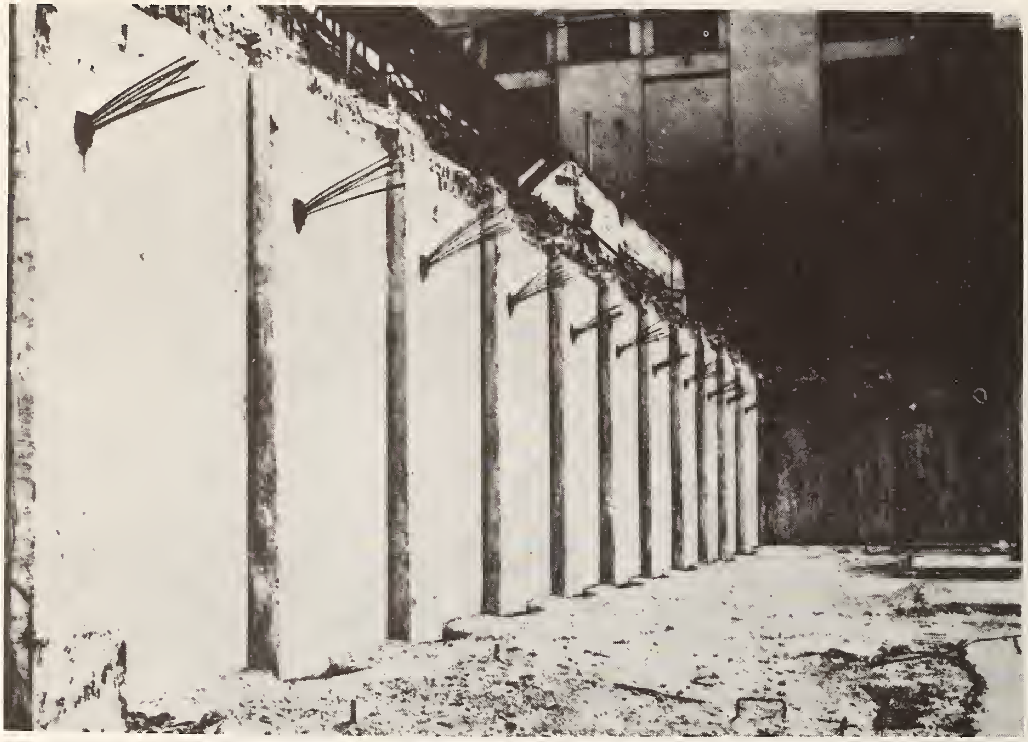
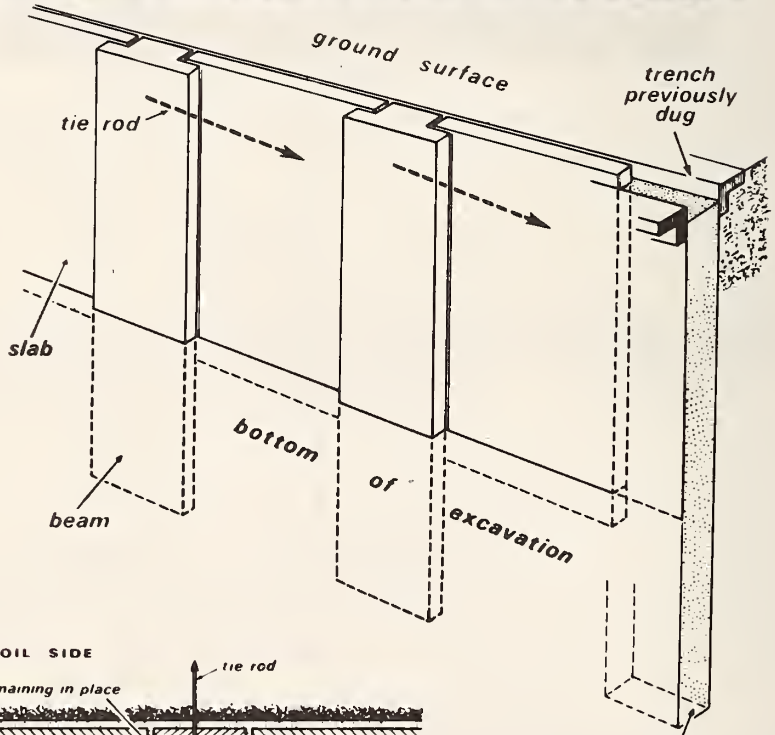


FIGURE 16



BEAM & PANEL WALL



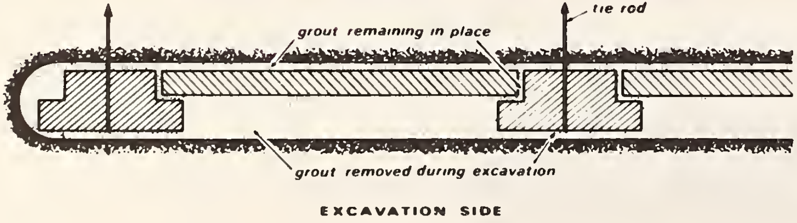
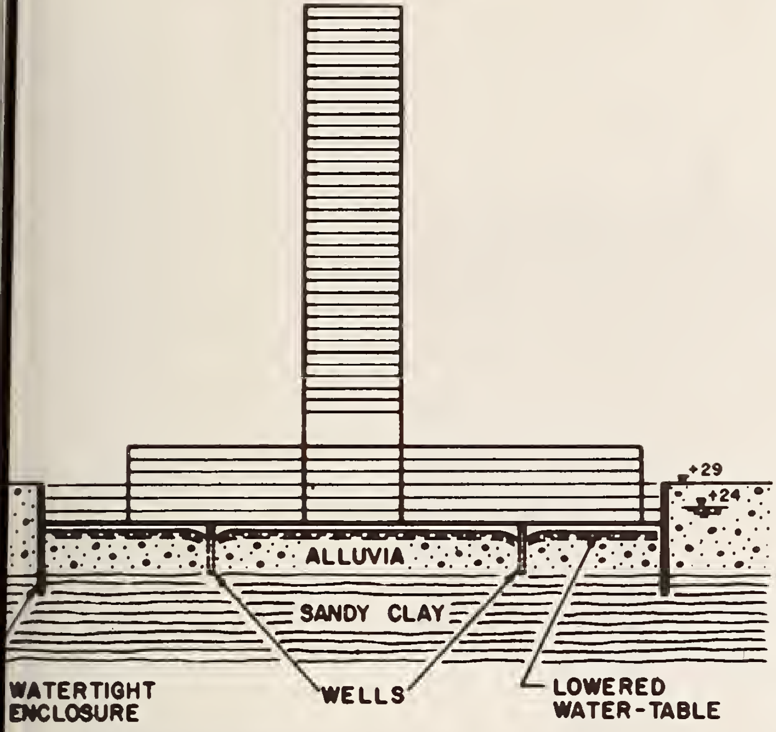


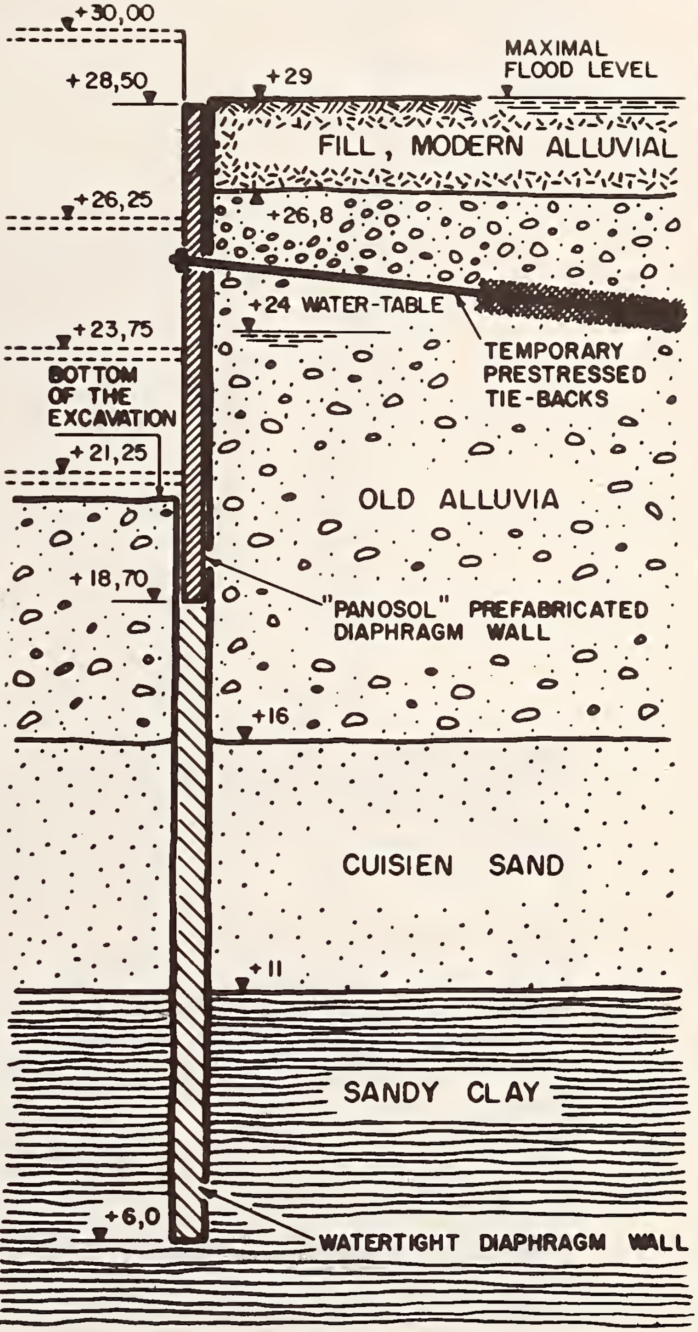
FIGURE 17



GROUND WATER CONTROL

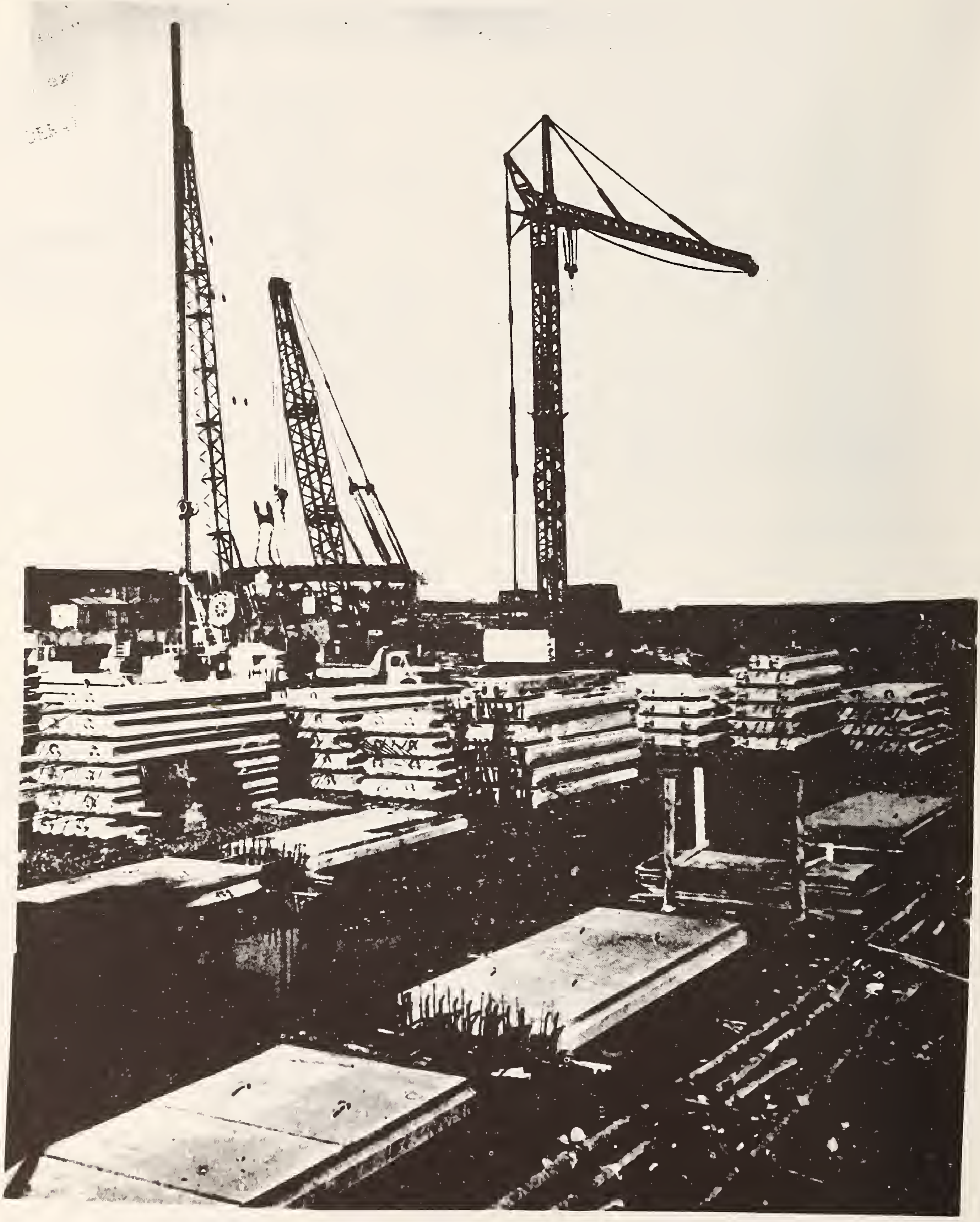
CONSTRUCTION DEWATERING

PERMANENT DEWATERING
TO REDUCE UPLIFT



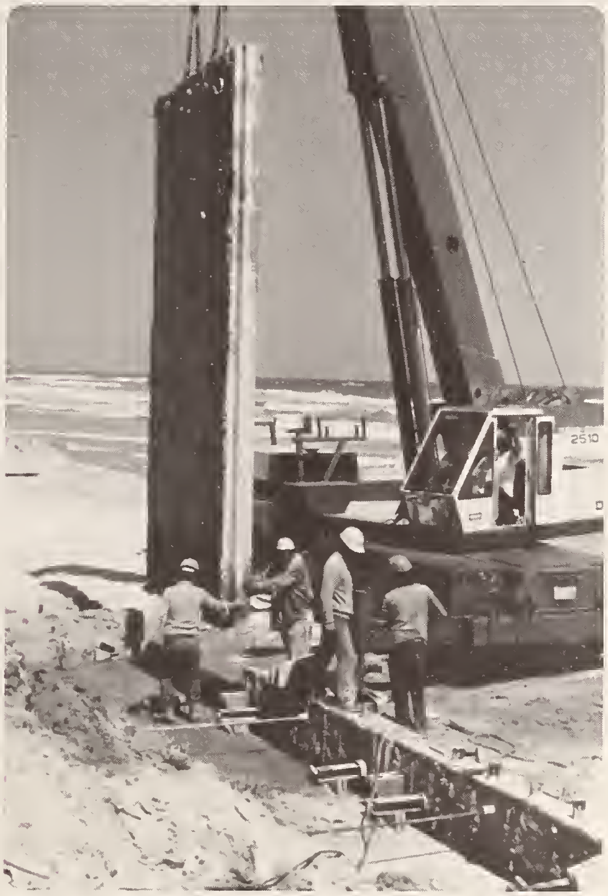
COMPOSITE PANOSOL WALL FOR LATERAL
SUPPORT AND GROUND WATER CUT-OFF

FIGURE 18



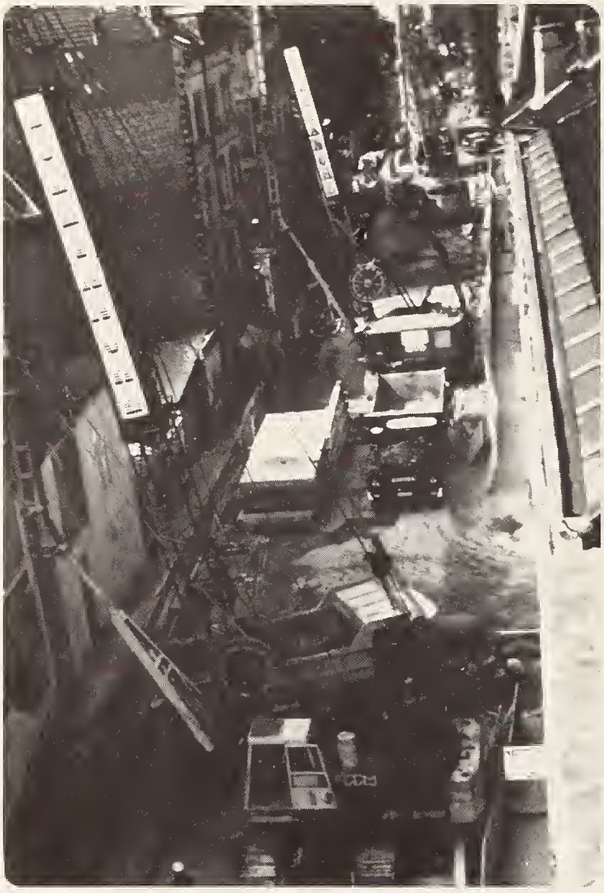
PRECASTING YARD FOR PANOSOL PANELS

FIGURE 19



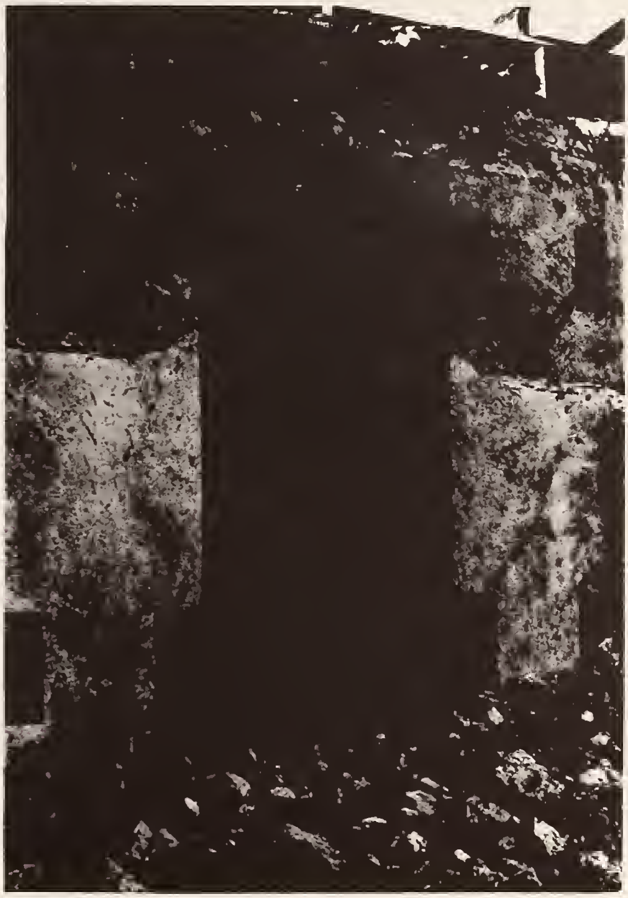
PANOSOL ELEMENT BEING
POSITIONED IN TRENCH

FIGURE 20



PANOSOL SITE WHERE "UNDER THE ROOF" METHOD IS BEING USED FOR DOUBLE BOX SUBWAY CONSTRUCTION

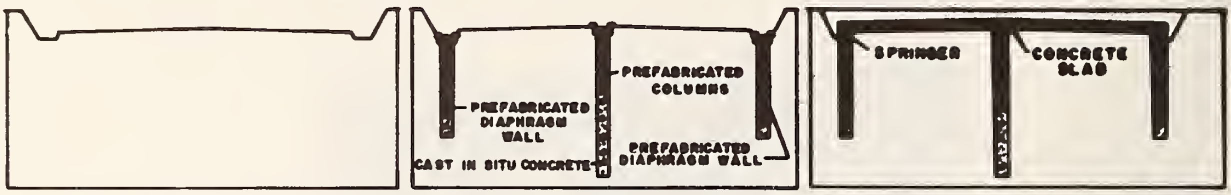
FIGURE 21



SECTION OF PANOSOL WALL INCORPORATING VARIOUS RESERVATIONS FOR TIE BACKS, FLOOR SLABS AND KEYS. CENTRAL PANEL HAS MOSAIC TILE FINISH

FIGURE 22

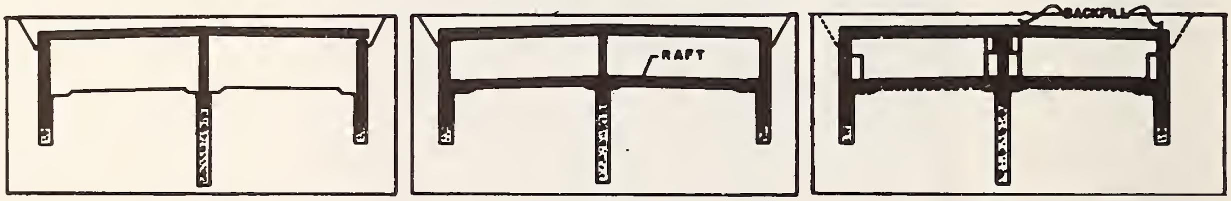
EXAMPLES OF THE "UNDER THE ROOF" CONTRUCTION METHOD



-1-
EXCAVATING FOR THE WORKING PLATFORM

-2-
PLACING THE PREFABRICATED WALLS

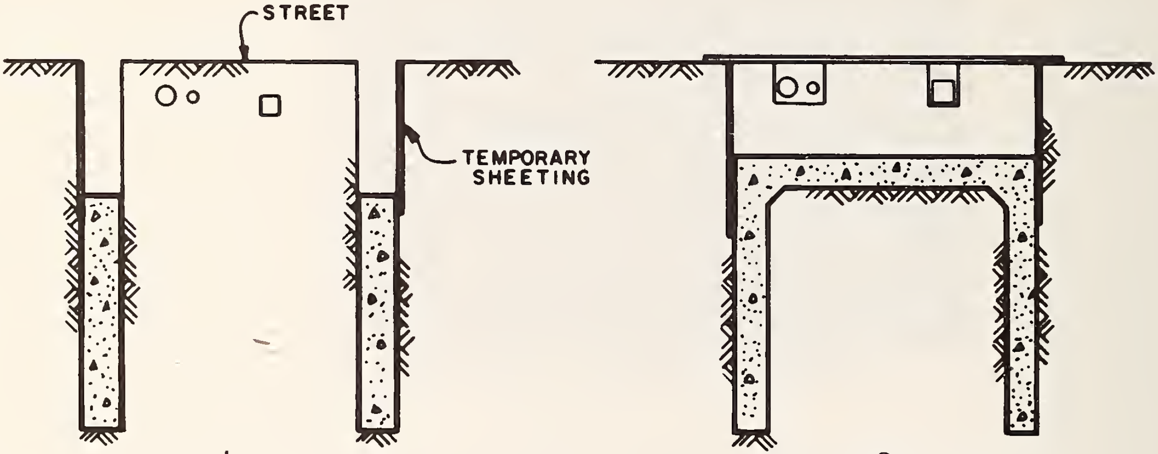
-3-
EXECUTING THE SPRINGERS AND SLAB



-4-
CARRYING OUT THE EARTHWORKS UNDER THE COVERING SLAB

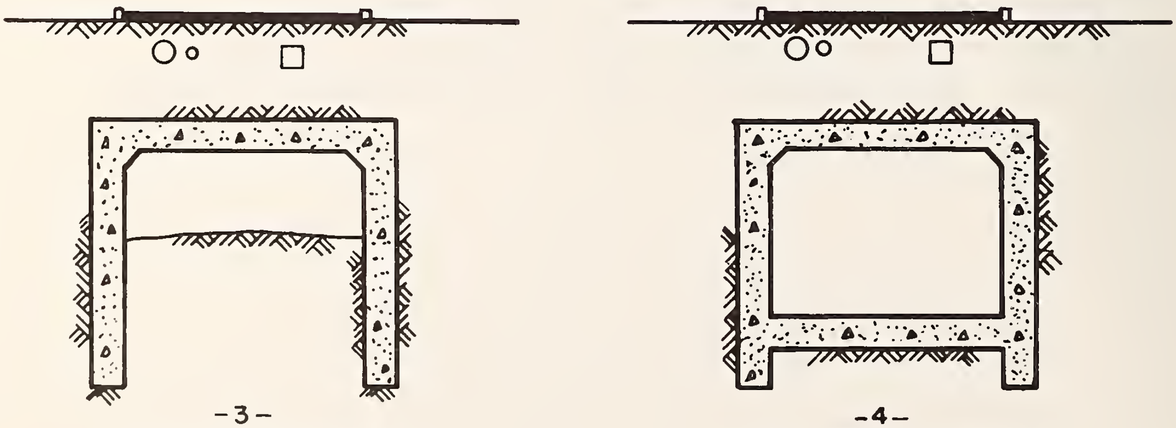
-5-
EXECUTING OF THE RAFT

-6-
END OF THE WORKS



-1-
CONSTRUCT SIDE WALLS FROM STREET LEVEL

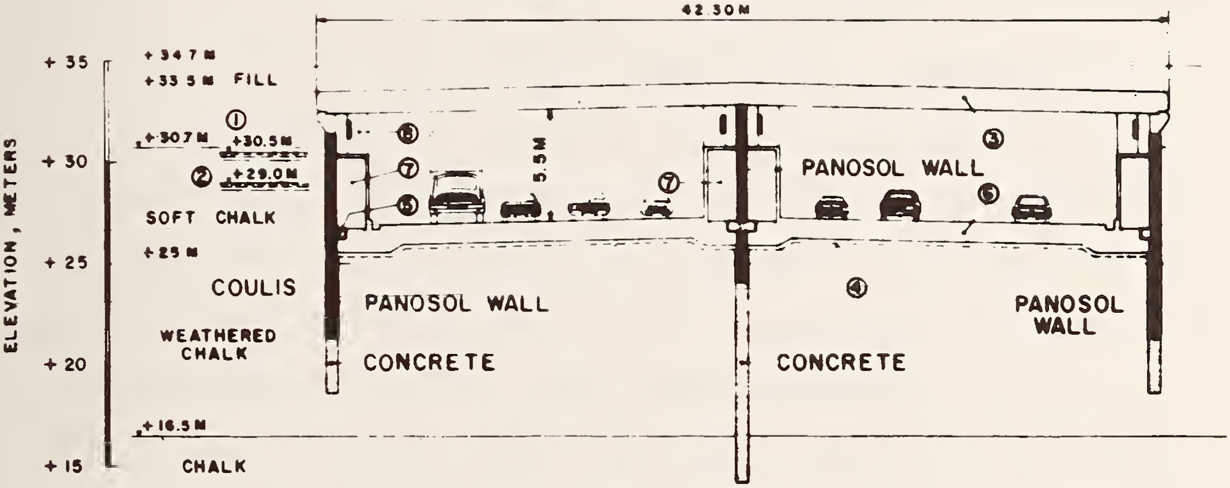
-2-
EXCAVATE & PLACE ROOF SLAB



-3-
RESTORE STREET WHILE EXCAVATING UNDER ROOF

-4-
COMPLETE EXCAVATION & POUR BASE SLAB

FIGURE 23



① DESIGN WATER TABLE
② CONSTRUCTION WATER TABLE
③ CONCRETE SLAB
④ BOTTOM OF EXCAVATION
⑤ STRUCTURAL KEY
⑥ BASE MAT
⑦ VENTILATION GALLERY
⑧ LIGHTING GALLERY

EXTENSION OF HIGHWAY A13 WHERE PANOSOL WALLS WERE USED IN CONNECTION WITH THE "UNDER THE ROOF" CONSTRUCTION METHOD

FIGURE 24

II. ELEMENTS OF DIAPHRAGM WALL DESIGN

In general, the design of a diaphragm wall involves the same types of considerations and criteria controlling design of conventional construction. This section of the discussion highlights three elements of the design: earth pressures acting on the wall, movements of the adjacent ground associated with excavation, and watertightness of diaphragm walls.

EARTH PRESSURE CONSIDERATIONS

When designing diaphragm walls for use in the permanent structure, two loading conditions must be considered: the temporary loading condition existing during excavation and construction, and the permanent loading condition existing after the structure is completed. The permanent loading condition is essentially the same as for conventional construction involving temporary sheeting and formed walls. However, the temporary loading condition existing during construction usually results in higher bending moments because of the large spans between the temporary supports and hence, frequently controls the overall design.

The construction loading conditions depend on construction procedures and especially the method used to support the walls during excavation. A wall supported internally by steel struts behaves considerably different than a wall supported externally by tie-backs. The characteristic pattern of movement of the wall is different between the two cases and also the distribution of earth pressures acting on the wall is different. An internal strut that is properly installed and wedged creates a reaction, fixing the point so that very little additional movement can occur at the strut level. On the other hand, a tie-back must undergo considerable movement in order for the load to change (the area of steel on a tie-back is small compared to the area of steel for the same capacity strut). Thus, a tie-back does not actually provide a reaction, but rather a fixed force. As an idealization, an internal strut provides a point of zero additional movement and a prestressed tie-back provides a constant force at its point of application.

The consequences of this difference in rigidity between struts and tie-backs are illustrated in Figure 1. The soil outside a struted excavation develops an arching active condition that is quite different from the conditions of movement and stress in the soil behind a wall supported by tie-backs. An arching active condition leads to a trapazoidal stress distribution as shown in Figure 1. For example, the Terzaghi and Peck design distribution has been developed from empirical data on walls supported with internal bracing. Comprehensive measurements made on a number of recently made struted excavations have shown the trapazoidal distribution to be a reasonable basis for design. The distribution of earth pressures on walls supported by tie-backs is quite different in that the earth pressures tend to be more triangular, corresponding to an earth pressure coefficient between Rankine active and the earth pressure at rest.

The triangular distribution results from the general absence of arching. Consequently, the bending moments in the wall and the elevation of the supports, should be different for strutted excavations and tie-back excavations. For strutted excavations, the vertical spacing between struts should be more or less constant. To optimize the design of a tie-back excavation and to minimize movements, more tie-backs are needed near the base of the excavation than near the top of the cut.

The pattern of wall movement, provided that bottom stability is not critical, is also quite different for the two cases. For a well made strutted excavation, the maximum movement generally occurs at the current excavation level or just below as the excavation proceeds. A properly placed strut fixes the movement at the excavation level so that further excavation causes little additional movement at the level of the strut. Consequently, the typical distribution of inward sheeting movement is approximately as shown in Figure 1 and corresponds roughly to translation and rotation about a point near the top of the wall. The point of maximum ground settlement occurs at a point some distance from the excavation. For a wall supported by tie-backs, the wall tends to translate laterally and rotate about a point near the bottom of the wall. Thus, the maximum inward movement tends to occur near the top of the wall rather than the bottom and the maximum ground surface settlement occurs near the edge of the cut.

Figures 2 and 3 illustrate the method best situated for determining design tie-back loads and bending moments in the wall as excavation proceeds. The method employs a triangular earth pressure distribution and considers the tie-back load and bending moments for each stage of excavation. The wall should be designed for the envelope of the various moment diagrams and the tie-backs should be prestressed to the maximum computed tie-back load.

The characteristic movements associated with the external stability failure mechanisms of internally braced and tied back excavations are shown in Figure 4. Again, the mechanisms are quite different. For a strutted excavation external stability is related to bottom stability, which is a problem only in weak soils and in excavations where bottom instability is caused by adverse seepage conditions. A bottom stability failure occurs when sufficient passive resistance cannot be mobilized to balance the active thrust on the wall, the bottom heaves and the walls move inward below excavation level. Bottom instability can also occur with a tie-back wall, however, a stability failure of the types shown in Figure 4, due to insufficient tie-back length must also be considered. With a strutted excavation the two walls do not act independently because they are connected by the struts. The lateral force on one side must be resisted by the lateral force on the other side. If the struts can sustain the lateral force, an internal failure will not occur. With a tie-back excavation, not only is adequate tie-back capacity required but the capacity must be generated far enough away from the wall to assure that stability failure of the wall·soil system cannot occur.

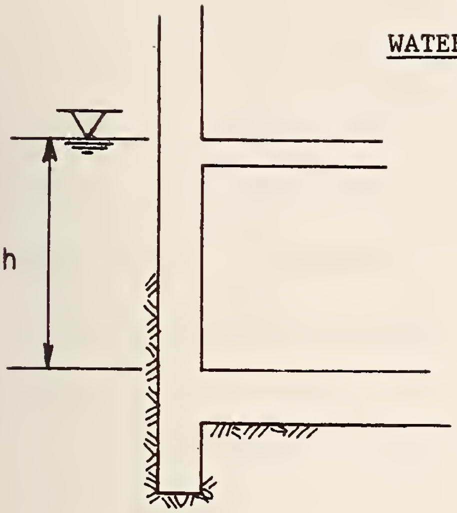
Figure 5 shows results of a stability analysis conducted using a rotational failure mechanism. To obtain a factor of safety of 1.2 for the support system, the anchorage zone for the tie-backs must be outside the circle corresponding to F=1.2. The practice of placing the anchorage zone just beyond the Rankine active zone as shown in Figure 6, leads to a factor of sefety against stability failure which is only slightly greater than unity. For the condition shown, the free length of the tie-backs is too short, especially near the bottom of the excavation where there is, in effect, no free length and the tie-back is reacting almost directly on the wall. As a rule of thumb, the criteria shown in Figure 6 gives the required free length best satisfying both stability and movement considerations.

Both empirical and theoretical considerations show that the length of the tie-back has an important influence of the magnitude of wall movement. Within certain limits the longer the tie-back, the smaller the movement of the wall.

Another important point regarding tie-back design involves the inclination of the tie-back with respect to the wall. There is often an economic advantage in placing the tie at a steep angle so that it is grouted in better soil. However, this leads to considerable axial load in the wall which must be resisted by end bearing if the angle of inclination exceeds about 20°. If the wall does not have adequate bearing, it will settle as the excavation is advanced and frictional resistance mobilized on the inside face of the wall is removed. Settlement of the wall will be accompanied by significant rotation and inward movement.

WATERTIGHTNESS

A summary of the Soletanche experience on watertightness of diaphragm walls is given in Table 1. The generalizations made in the table presuppose that flaws in the wall or joints due to poor concreting or poor cleaning of the joint have been corrected. If the permeability of the soil is less than the permeability of the concrete, dampness or wet spots on the wall are almost never encountered regardless of the total head acting on the wall, provided there is some ventilation in the basement. For cast-in-place concrete walls, experience has been that if h, the height of the water table above the basement slab, is less than about 15 feet, there is no dampness, again, provided there is some minimal ventilation. When height is more than about 15 feet, there may be humidity on both the joint and the concrete. The ordinary solutions for eliminating the humitity are indicated in the table. One solution is to roughen the concrete and coat the wall with epoxy or resin.



WATERTIGHTNESS OF DIAPHRAGM WALLS

$k_{soil} < k_{concrete}$, no dampness regardless of h provided that there is minimal ventilation

Cast-In-Place Concrete

h<15 ft	No dampness - repair bad joints during construction - provide minimal ventilation
h>15 ft	May have humidity on both joint and concrete - roughen concrete and spray or paint resin - 2-3 " mortar (place in 1/2" layers)

Panosol

h<20-25 ft	No dampness on wall (because of grout and better concrete)
h>25 ft	Place bitumen coating in soil side of wall

Joint must be treated regardless of h

Table 1

Another solution is to place several inches of layered mortar on the inside face of the wall. With PANOSOL, there is no dampness on the wall for h less than about 20 or 25 feet, because of the grout between the soil and the wall and because the concrete is denser. When h exceeds about 25 feet, a bitumen coating on the soil side of the panel or special additives to improve the watertightness of the concrete are advisable. Regardless of the head of water, the PANOSOL joint should be treated.

The above procedures are designed to prevent dampness. If a diaphragm wall is properly constructed, water droplets should never form on the inside face of the wall due to seepage.

GROUND MOVEMENTS ASSOCIATED WITH EXCAVATION

Excavation removes a mass of soil and water and produces a reduction in total stress along the sides and bottom of the cut. When necessary, the ground water is lowered to facilitate the removal of soil and construction inside the excavation. The reduction in total stress during excavation causes the soil to move inward toward the excavation and upward at the bottom of the cut. The upward movement of the bottom is accompanied by an inward movement of the soil below excavation level. As these movements occur, the soil adjacent to the excavation undergoes lateral displacement and settlement.

A large number of interrelated factors influence the magnitude and distribution of movements accompanying excavation. Movements depend on the dimensions, and especially the depth, of the excavation; the soil conditions; the rigidity and the method and sequence of installing the support system; the time that the excavation is left open; and the construction details and workmanship. The method of ground water control also has an important influence on movement.

Installation of lateral bracing or tie-backs is always preceded by excavation to the support level. Once excavation has passed a given elevation, further inward movement above that elevation is controlled entirely by the bracing details, i.e., the vertical and horizontal brace spacing, the brace prestress, the elapsed time before the brace is wedged into position, and the quality of workmanship. Movements occurring below excavation level are determined by the combined resistance to inward movement provided by the embedded portion of the wall extends to a great enough depth below the bottom of the cut and is stiff enough to resist the bending moment caused by the soil stress acting on the outside face of the wall, then lateral movements of the soil outside the excavation will not occur regardless of the properties of the soil. As a practical matter, conventional sheeting rarely posseses the required rigidity; on the other hand, diaphragm walls may have sufficient rigidity depending, of course on their design. Consequently, for conventional sheeting systems, movement depends chiefly on the properties of the soil; if the soil immediately below the base of the excavation must undergo large strains to develop passive resistance or if the soil cannot develop sufficient passive resistance to counterbalance the active thrust, then the inward movement of the soil toward the excavation and the resulting ground settlement will be large.

Movements Related to Soil Type

Figure 7 summarizes data from the engineering literature on the maximum movements measured during excavation in various types of soils. Different symbols are used to denote case studies involving conventional soldier pile or sheeting systems and diaphragm walls.

For excavations made in cohesionless sand and cohesive granular soil the available studies suggest that maximum movements are relatively small and typically on the order of 0.1 to 0.2% of the excavation depth, provided that the ground water level is below the bottom of the excavation or otherwise brought under complete control. Ground water control in these types of soil is very important as demonstrated by the following examples. Figure 8 shows a case where large movements took place in a cut supported by sheeting because the ground water level was not lowered sufficiently. The case involved a subway excavation in New York made in fine sand and coarse silt close to buildings founded in short piles. Sheet piling was driven along the sides of the cut and ground water flowed into the excavation by percolating up into the excavation from below the bottom of the sheet piles. These conditions lead to a reduction in effective stress in the soil at the base of the excavation, a consequent loss of passive resistance below the base of the cut. Large inward movement of the sheeting resulted, causing footings of the adjacent buildings to settle about 6 inches. Another case involves a subway excavation in Boston. The soil profile and movements are shown in Figure 9. The top layer is fill, a loose mixture of gravel, sand, silt and clay. Below the fill is a layer of compressible organic silt containing shells. The till is quite erratic and consists of stratified layers of sandy clay and dense sand and gravel. Bedrock is a hard jointed volcanic intrusion. Seepage into the excavation was collected in sumps and pumped out of the hole. Because of the compressible organic silt and the presence of important nearby structures, the contractor was required to maintain the total head in the till outside the excavation at or above +95 ft. The static value of total head prior to construction was about +110 ft. As shown by the piezometer measurements, the contractor was not able to meet this specification even though an extensive effort was made to recharge the till. About two-thirds of the settlement was due to consolidation of the organic silt caused by the reduction in pore pressure. The settlements severely damaged the adjacent building.

Movements of excavations made in soft and medium clays can be quite large. Maximum inward movements of the sheeting and maximum surface settlements are commonly on the order of 1.0 to 2.0% of the excavation depth when conventional sheeting systems are used. The data shown in Figure 7 indicates that movements are reduced by a factor of 5 to 10 when diaphragm walls are used.

As the stiffness of the clay increases, the movements caused by excavation decrease rapidly. Only a small number of cases have been reported for clays whose undrained shear strength is greater than about 2000 psf. For these clays maximum movements are only a small fraction of one percent of the excavation depth.

Measurements made at several test sections along a subway excavation in Boston have demonstrated that movements associated with excavation in compressible organic silt can be as large as the movements caused by excavation in soft clay.

Influence of Bracing and Wall Stiffness on Movements

The relationship among settlement, sheeting movement and bottom heave has been demonstrated by comprehensive measurements made at several excavations in Oslo. The manner in which the ground settlement and the sheeting movements developed during one excavation is shown in Figure 10. The volume of settlement of the adjacent ground was approximately equal to the volume of inward movement of the sheeting. Sheeting movements were related to the heave measured below excavation level.

Figure 11 illustrates in a qualitative manner the stress changes and strains experienced by two soil elements near an excavation. The stress paths are typical for a normally consolidated plastic clay.

There are a number of design and construction decisions or alternatives that have an important influence on the movements caused by excavation. These factors can be grouped into two categories: factors that have a primary influence on sheeting movements above excavation level and factors that influence sheeting movements below excavation level.

Lateral wall movements that occur above excavation level are determined by the procedures and details associated with brace installation. Some of the important factors controlling these movements are listed in Table 2.

If horizontal movements are measured at frequent intervals during excavation, the movements occurring above and below excavation level as the depth of the cut is advanced can be determined quite accurately. Figure 12 shows sheeting movements measured at verious stages of excavation in a medium clay in Boston. The solid curve is the final sheeting movement after excavation was complete. The shaded area under this curve is the amount of movement that took place below excavation level; the unshaded area represents the movement that took place above excavation level. For example, at the depth of 40 ft, about 2 inches of inward movement took place before the excavation level reached this depth, while about 3 inches of additional movement occurred when the excavation level was below 40 ft. The maximum excavation depth was 47 ft so, all the movement below this depth occurred below excavation level.

Figure 13 compares the deflection area occurring above excavation level to the total deflection area of the sheeting at the end of excavation

FACTORS CONTROLLING LATERAL WALL MOVEMENTS THAT OCCUR ABOVE EXCAVATION LEVEL

Horizontal and Vertical Brace Spacing

Depth of Excavation Below Brace Level Before Brace Is Installed

Length of Excavation Parallel to Wall Made at Any One Level Prior to Installing Braces at That Level

Elapsed Time Between Excavation and Brace Installation

Details of Prestressing and Wedging Braces

Details of Excavating and Placing Lagging Between Soldier Beams

Table 2

for a number of cases where sheeting movements were monitored on strutted excavations. The average vertical strut spacing varied from about 2 to 3.5 meters. For these cases, which probably represent good to excellent construction practice, 20 to 40% of the total deflection area occurred above excavation level. When less attention is given to the excavation and bracing details, a much greater percentage of the total wall movement may occur above excavation level.

These points are further emphasized by Figure 14 which compares sheeting movements measured at two nearby sections of subway excavation in Boston. Section B had a deeper excavation than Section A, but the thickness of the compressible soil was greater at Section A. The excavation at Section B was made first and several improvements were made in the excavation and bracing procedures at Section A. These improvements involved greater control over the depth and length of excavation prior to installing struts at each level, greater control over the elapsed time between excavating and installing struts, and improved prestressing and wedging procedures. As a result, the maximum sheeting movements at Section A were reduced by about a factor of two compared to the movements at Section B.

As discussed previously, the lateral movement occurring below excavation level is controlled by the combined resistance of the embedded portion of the wall and the soil below the base of the cut. Soldier piles placed at the normal spacing of 6 to 10 ft provide little resistance to overall movement below the excavation level. Experience has shown that even the heaviest sections of steel sheeting are not usually stiff enough to significantly restrict lateral movement. However, cast-in-place concrete walls constructed using the slurry trench method are much stiffer than steel sheeting and for this type of construction, available field data suggests that the rigidity of the wall has an important influence on limiting movements below excavation level.

Figure 15 presents a rough theoretical basis for determining the required wall strength to prevent large inelastic movement in cuts in cohesive soils. The model considers the element outside the wall to be in an active state and the element below the base of the cut to be in a passive state. The difference between the maximum passive resistance that can be mobilized and the active stress must be sustained by the wall, or else large inelastic movements will occur. The formula and curves presented in Figure 15 indicate that when the factor of safety against bottom stability failure is less than 1.5, i.e., when the total unit weight of the soil times the height of the excavation divided by the undrained shear strength is less than about 4, then the wall does not need to provide much reisistance below excavation level to prevent inelastic movement. In this case maximum movements rarely exceed 0.5% of the excavation depth. However, if the factor of sefety drops below 1.5, movements will increase substantially unless the wall can provide significant resistance.

Figure 16 presents a hypothetical example of an excavation in a profile having 80 feet of soft to medium clay. The profile could be in San Francisco, Boston, Detroit, Chicago, Cleveland, etc. There is sand fill near the surface and a high ground water level. The excavation depth is 50 ft. For this case the factor of safety against bottom stability failure is 1.0, and according to empirical correlations such as those shown in Figure 17, the expected movements for a conventionally sheeted excavation are on the order of 2 percent of the excavation depth or approximately 1 ft. These large movements can be prevented by constructing a wall with sufficient resistance below the base of the cut. Shown in Figure 16 is the stress that the wall must sustain as calculated from the analysis given in Figure 15. A rigid diaphragm wall 3 to 4 ft thick is required.

One case very similar to the preceeding example where a cast-in-place concrete wall greatly reduced movements was at the Embarcadero Station in San Francisco (Figure 18). The soil profile includes about 65 ft of soft bay clay. The cast-in-place concrete wall was 4.5 ft thick and reinforced with large steel beams. The wall extended through the clay and into an underlying deposit of dense sand and stiff sandy clay. Even though the factor of safety against bottom failure was about 0.6 to 0.8, the maximum measured wall deflections at the end of construction were on the order of 1 to 1.5 inches, or about 0.1 to 0.2% of the excavation depth. These movements are a factor of ten less than the movements indicated in Figure 17 for cases where N_{cb}/N_b is about 0.8 to 1.0.

A second case demonstrating the effectiveness of diaphragm walls is reducing movements involves the Boston subway excavation in medium clay shown in Figure 19. A cast-in-place concrete wall was used on a 200 ft long section near a seven story school building on shallow foundations. The school was very close to the alignment and it was more economical to use a diaphragm wall than to underpin the building. Adjacent sections were constructed using steel sheeting. The medium Boston blue clay is slightly over consolidated near the surface and the undrained shear strength actually decreases somewhat with depth from 1000 psf to 600 psf where the clay becomes normally consolidated. The excavation was 50 ft deep. The bottom stability number for the excavation was about 5 or 6 (factor of safety = 1.0 to 1.2) suggesting maximum movements for conventional sheeting in the range of 6 to 12 inches.

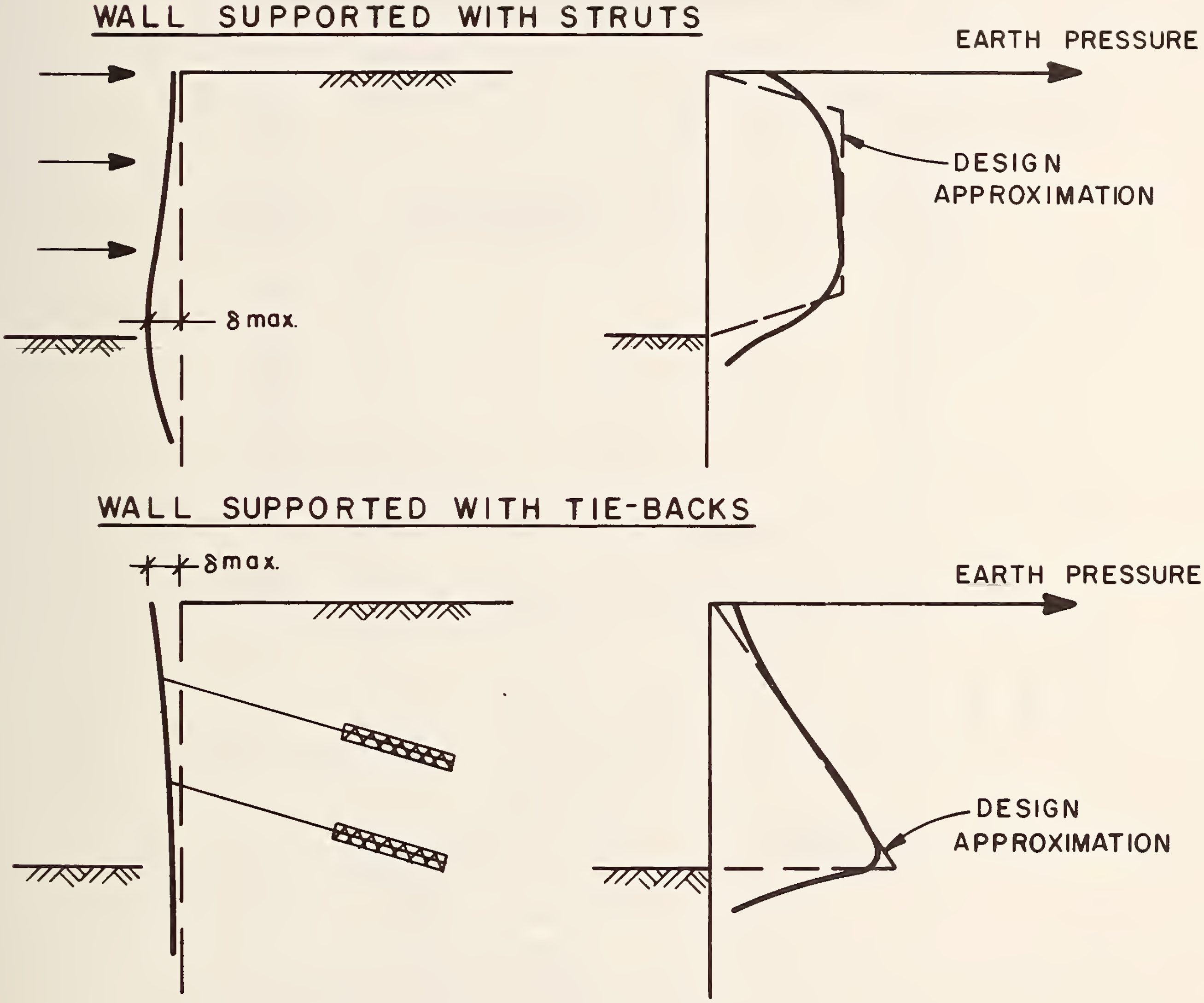
The soil profile, the wall depths and the brace spacings were nearly identical for both the sheeting section and the diaphragm wall section, the only significant difference was the wall stiffness. The lateral wall movements at the end of excavation are compared in Figure 19. Inclinometers placed in the concrete wall showed maximum lateral movements of about 0.5 inches as compared to maximum movements of 4.5 and 7 inches measured at the section using steel sheeting. In this case, where a direct comparison can be made, the diaphragm wall reduced maximum movements by a factor of 10 compared to conventional sheeting.

Figures 20, 21, and 22 are photographs showing a few examples of completed diaphragm walls constructed immediately adjacent to existing structures on shallow foundations that were not underpinned.

Summary

Ground movements associated with excavation are a common cause of damage to adjacent property. The largest movements and therefore the most serious problems, are associated with excavation in soft and medium clay and compressible silt. For major excavations in these soils, maximum ground settlements are commonly on the order of 1 to 2% of the excavation depth. On the other hand, excavation in granular cohesive soil and stiff clay usually causes maximum movements less than 0.2% of the excavation depth, in those cases where seepage pressures and ground water flow are brought under good control.

For excavation in clay, lateral wall movements are associated with settlements of about equal volume. Measurements on wall designed and constructed sheeting excavations show that 60 to 80% of the total lateral wall deflection takes place below excavation level. These movements depend on the passive resistance to inward movement that can be mobilized by the soil immediately below the base of the cut. When the excavation reaches a depth where bottom stability failure is approached, both the lateral movement and the ground settlement increase sharply. The bending resistance of conventional soldier piles or steel sheeting is not sufficient to restrict lateral wall movements occurring below excavation level. On the other hand, cast-in-place concrete walls constructed by the slurry trench method are much stiffer in bending than sheeting. As demonstrated in a number of recent projects, stiff walls of this type can reduce movements, even in soft clays, by about an order of magnitude compared to conventional sheeting.



DISTRIBUTION OF LATERAL EARTH STRESS AND MOVEMENT

FIGURE 1

METHOD FOR COMPUTING BENDING MOMENTS & TIE LOADS FOR TIE-BACK WALL

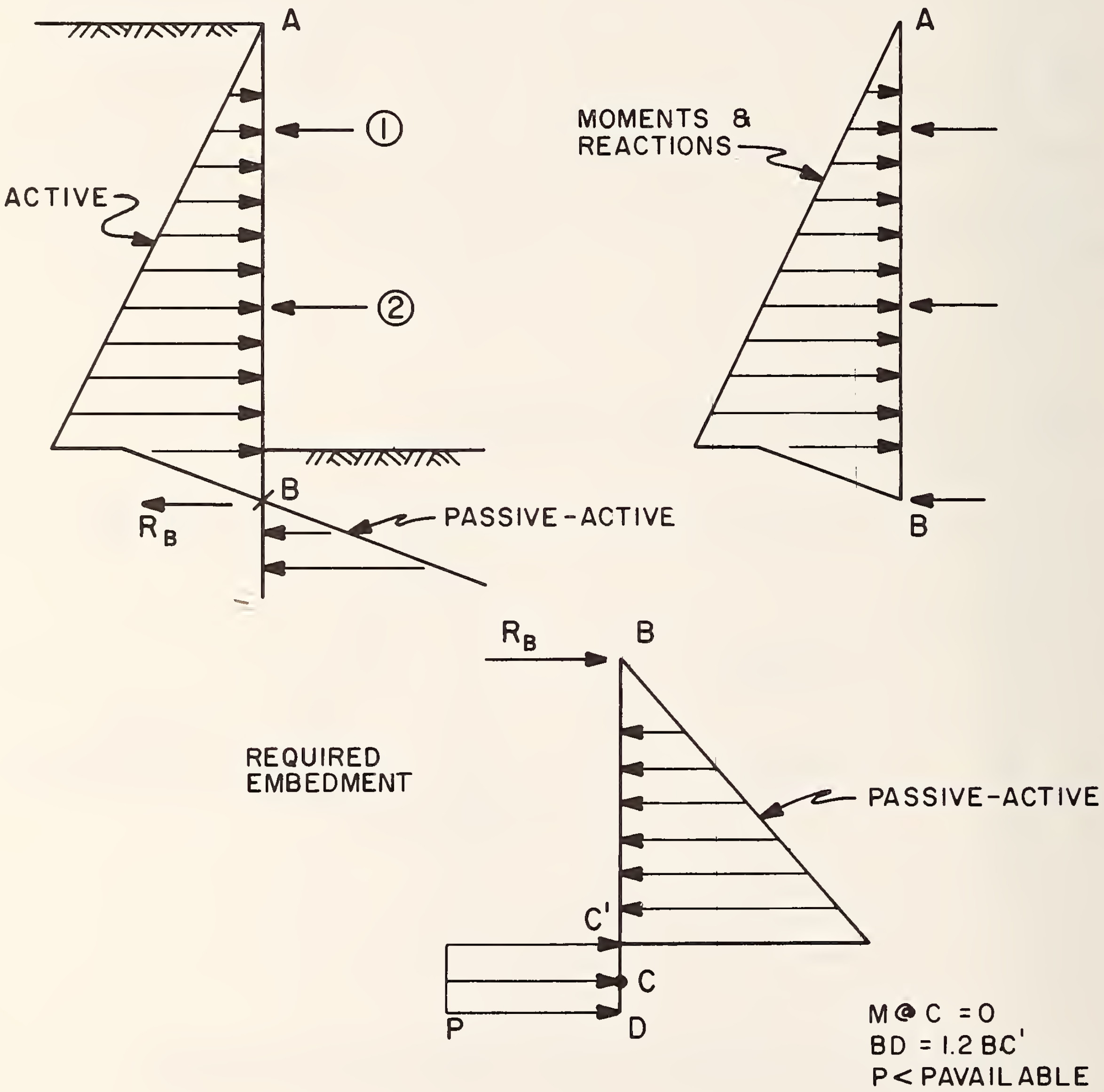
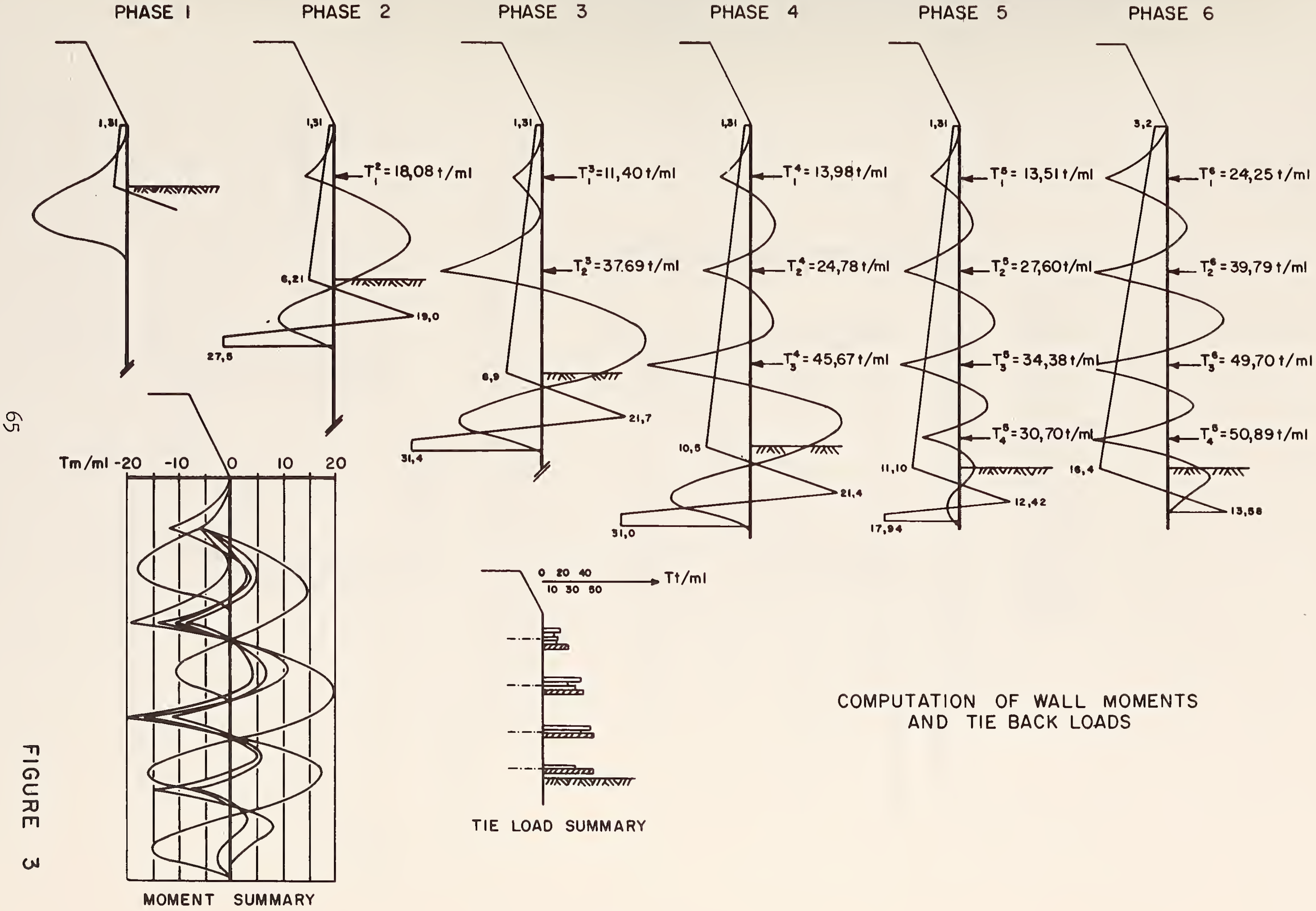


FIGURE 2

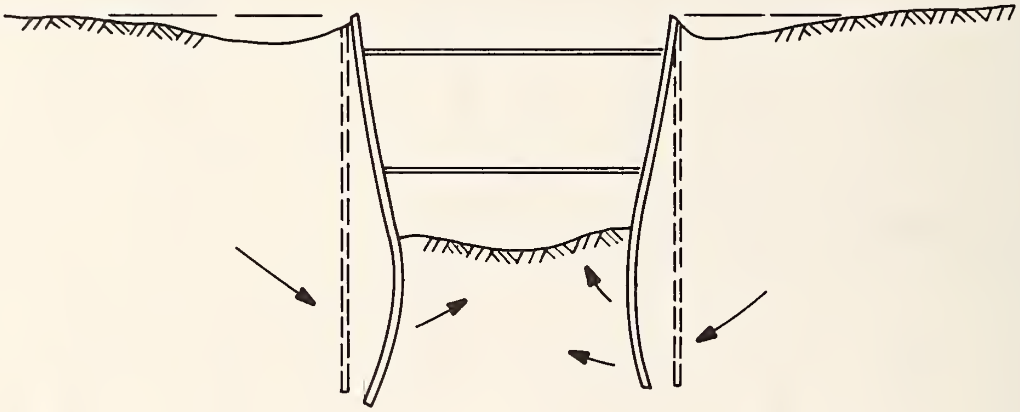


COMPUTATION OF WALL MOMENTS AND TIE BACK LOADS

FIGURE 3

MECHANISMS CONTROLLING OVERALL EXTERNAL STABILITY

INTERNAL STRUTS



TIE-BACKS

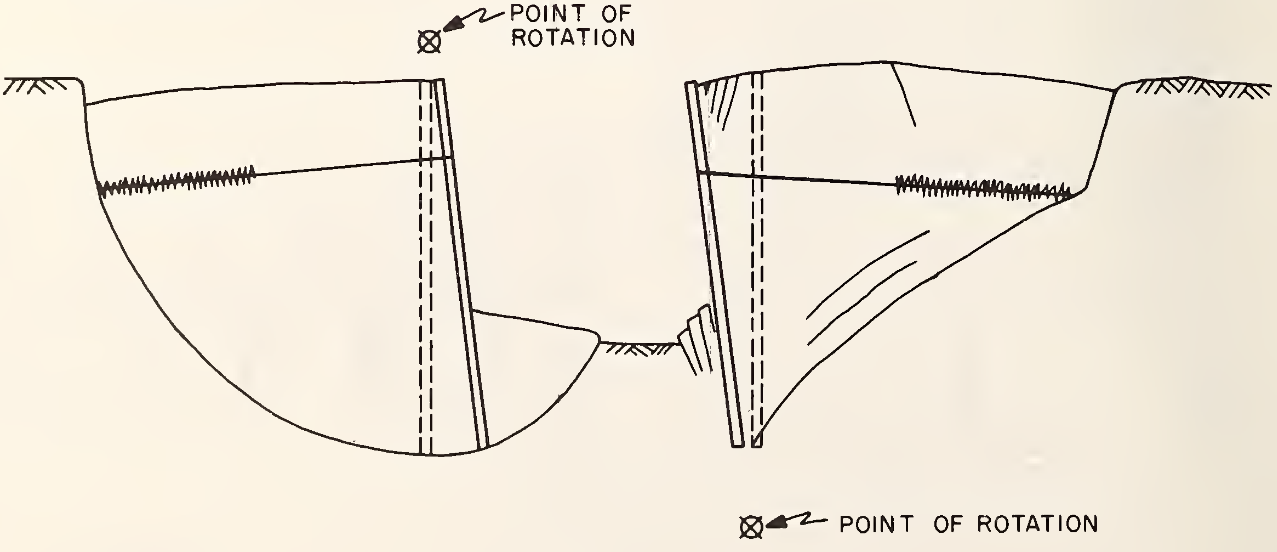
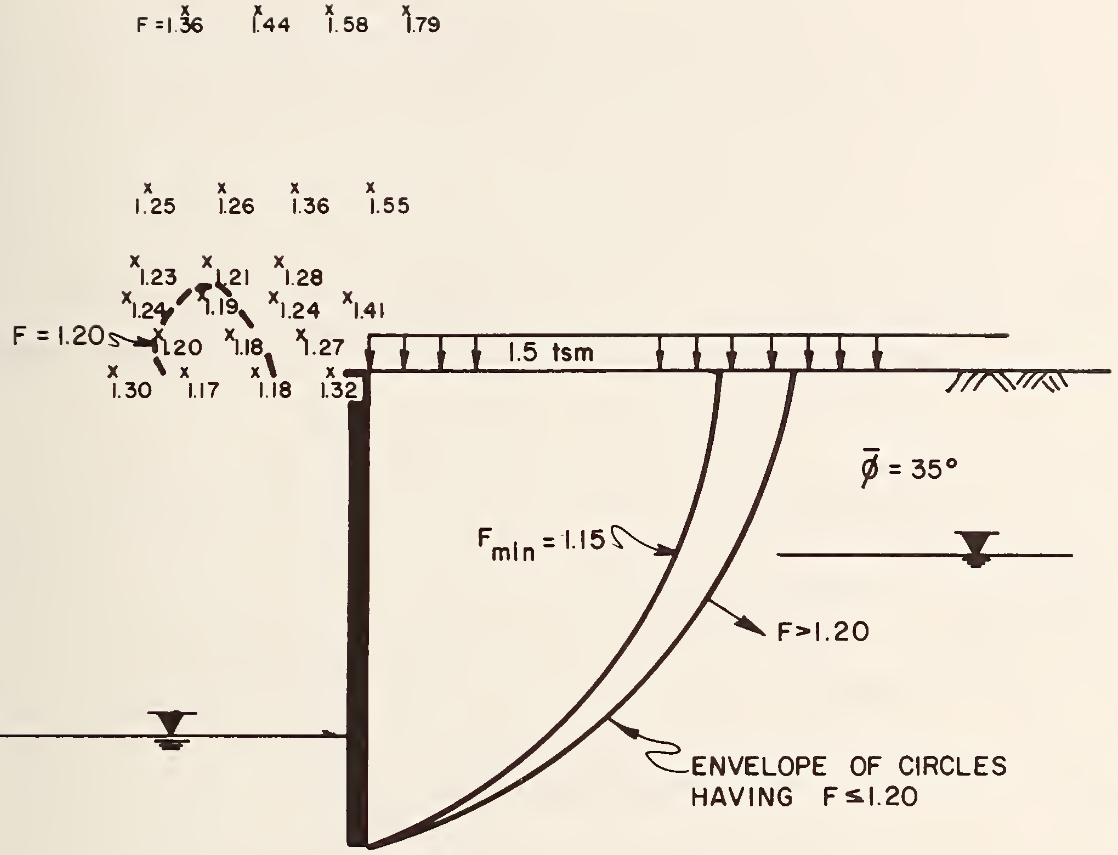
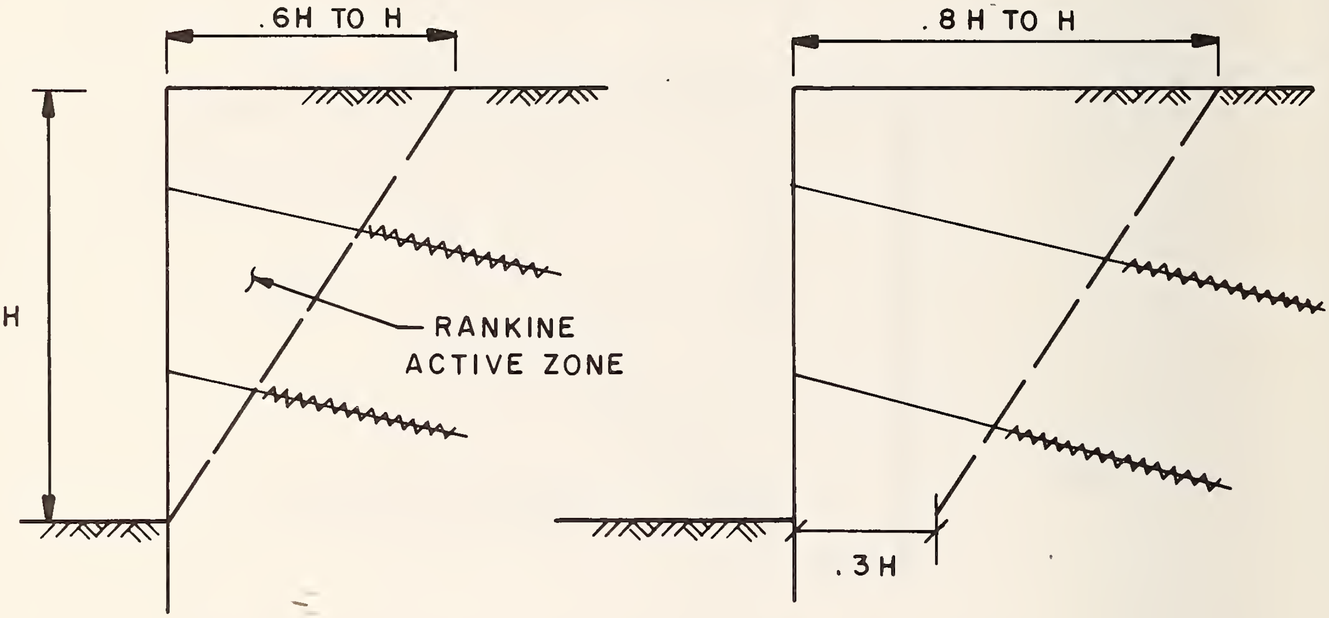


FIGURE 4



SUMMARY OF STABILITY ANALYSES
FOR CIRCULAR FAILURE SURFACES

FIGURE 5

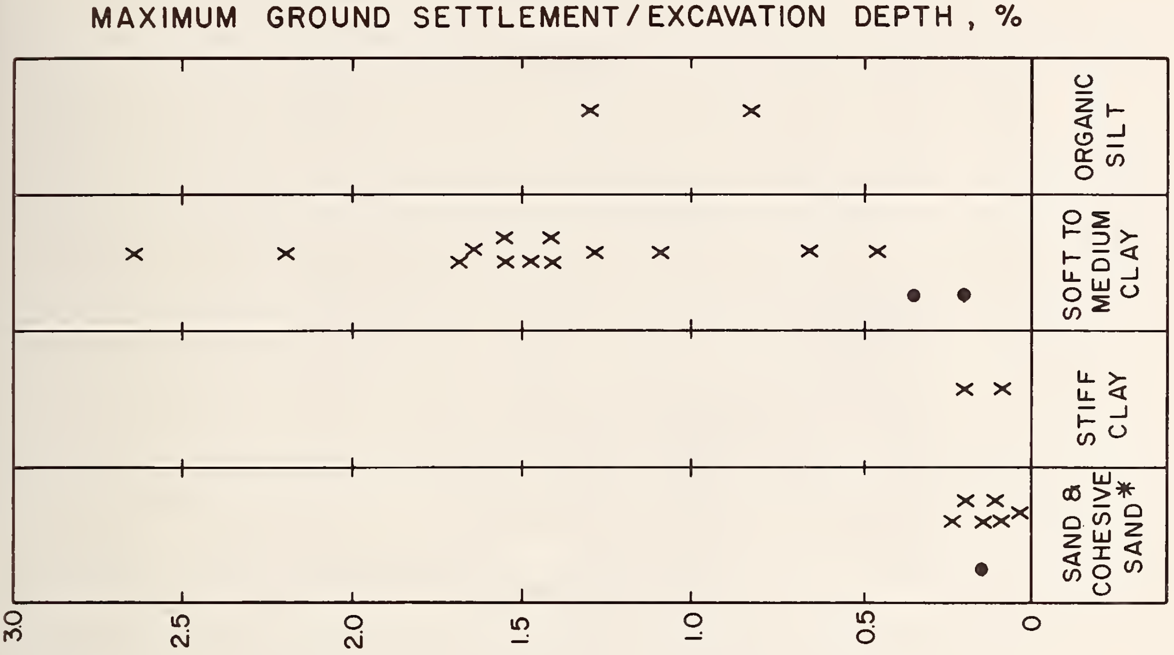


- NOT SUFFICIENT FOR GENERAL STABILITY
- NOT LONG ENOUGH TO MINIMIZE MOVEMENT

- USUALLY SATISFIES BOTH STABILITY AND MOVEMENT CONSIDERATIONS

FREE LENGTH CONSIDERATIONS
FOR TIE-BACK WALL

FIGURE 6



* DOES NOT INCLUDE CASES WHERE LARGE MOVEMENTS HAVE BEEN CAUSED BY SUBSURFACE EROSION OR PIPING

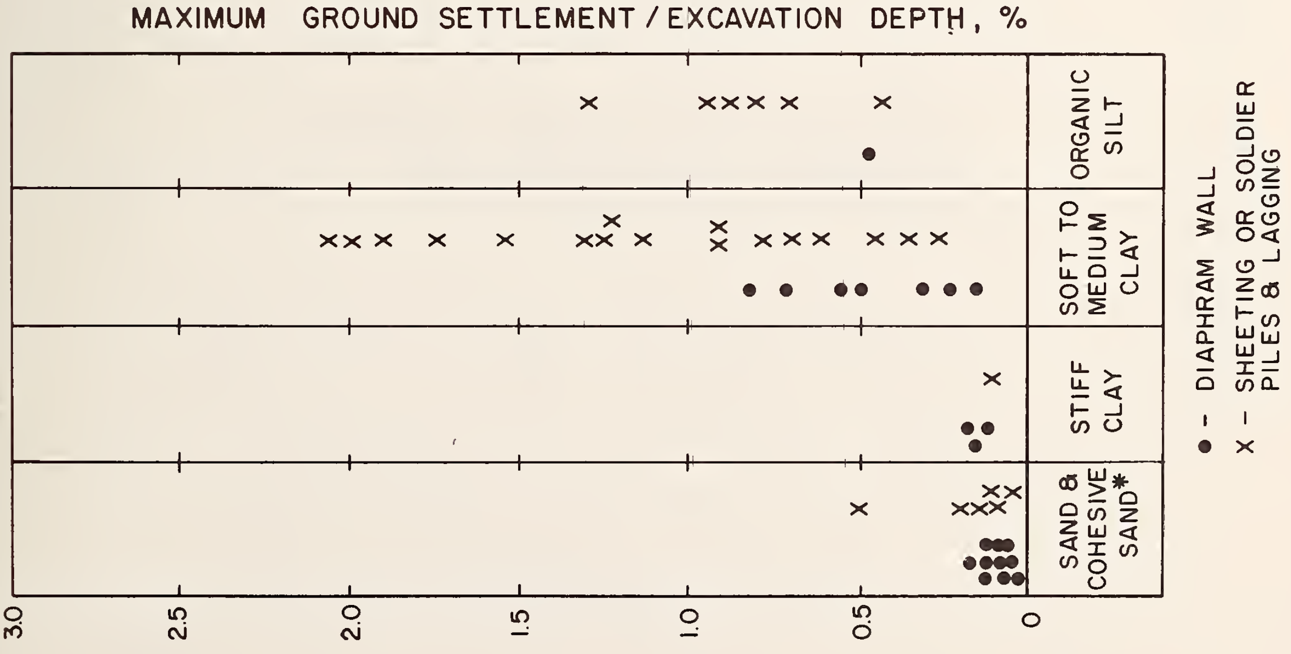
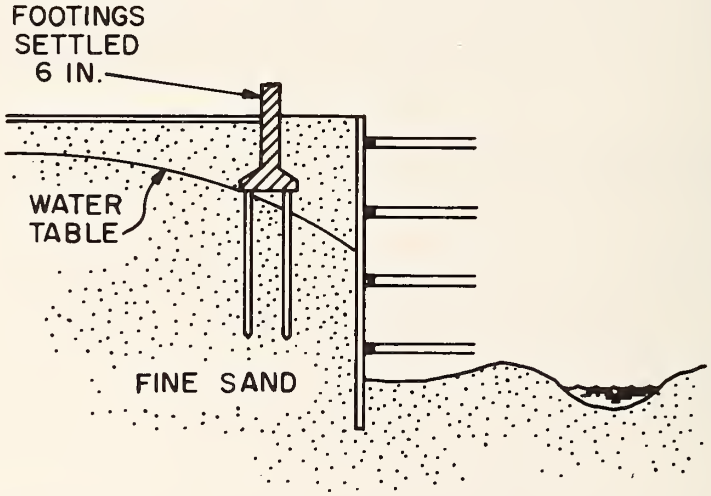
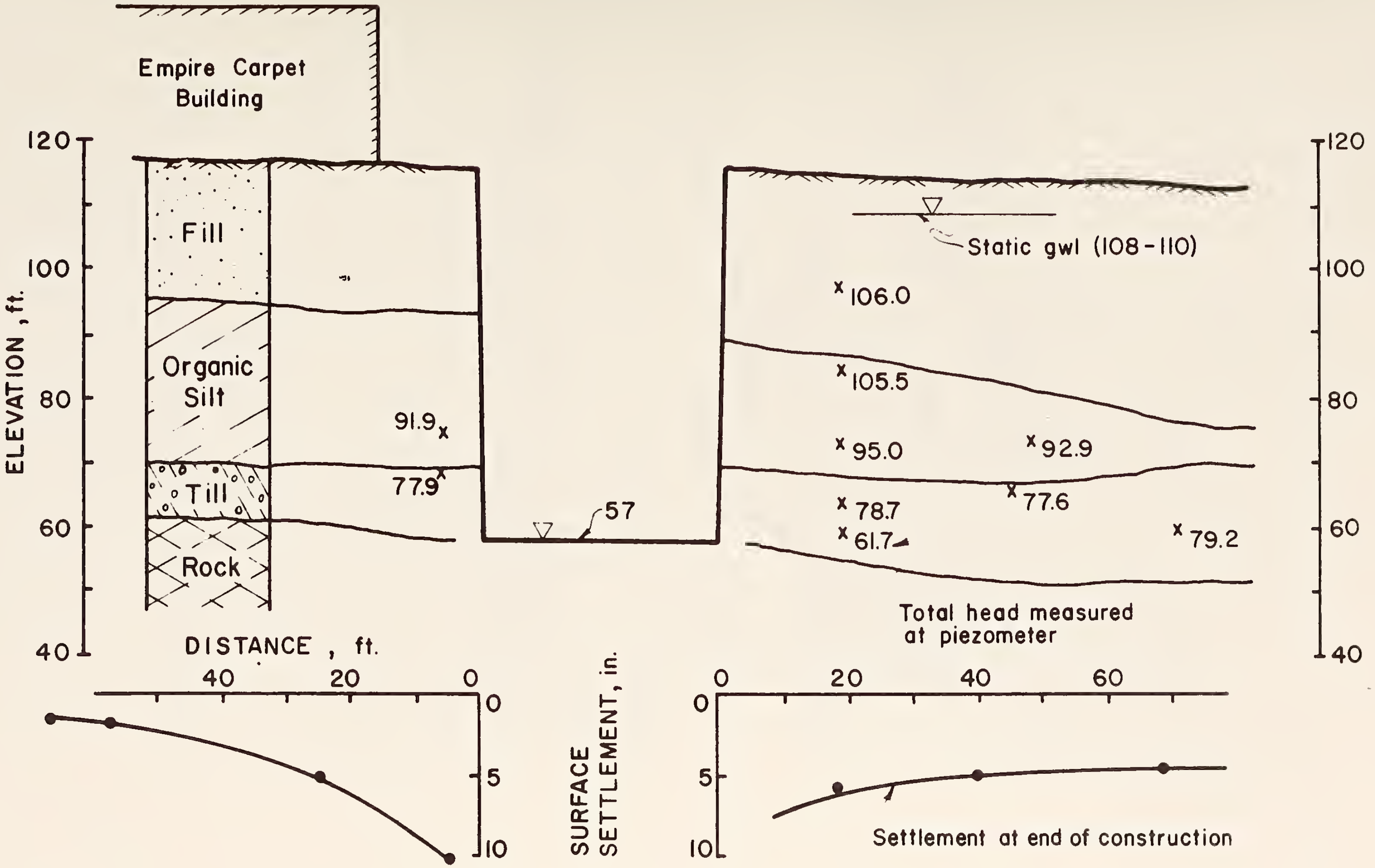


FIGURE 7



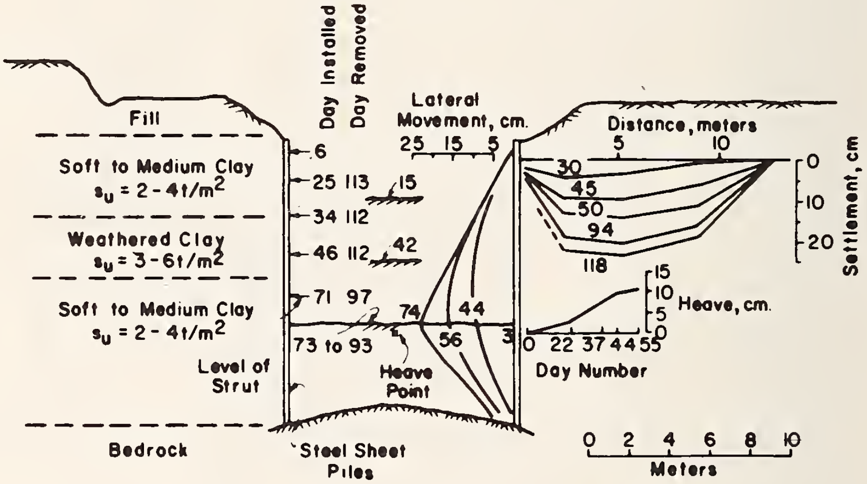
EXAMPLE OF CASE WHERE LARGE SETTLEMENTS OF BUILDING ADJACENT TO SHEETED EXCAVATION WERE DUE TO PIPING BY HEAVE

FIGURE 8



CROSS SECTION AT SUBWAY EXCAVATION IN BOSTON SHOWING CHANGES IN TOTAL HEAD AND SURFACE SETTLEMENT CAUSED BY EXCAVATION AND DEWATERING

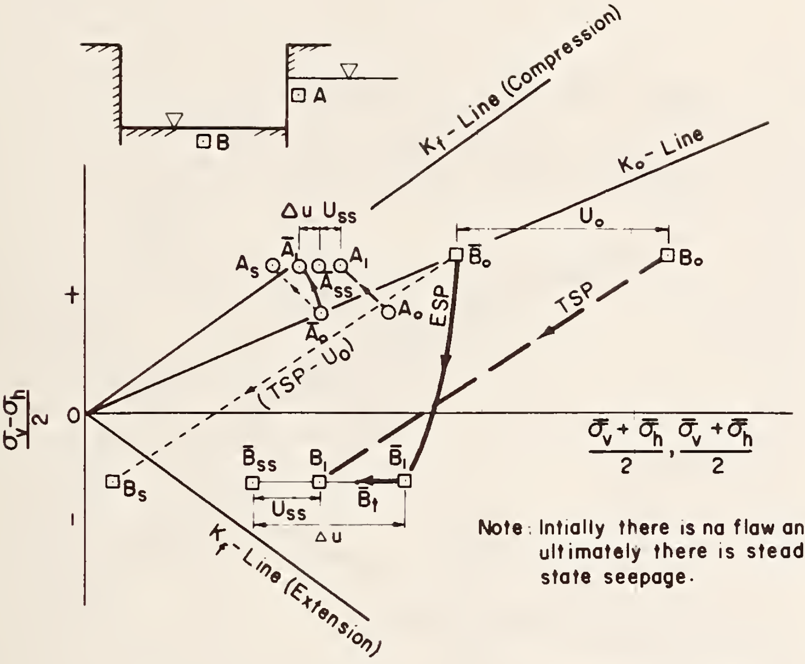
FIGURE 9



from Peck (1969)

RELATIONSHIP OF CONSTRUCTION PROGRESS TO MOVEMENTS OF OPEN CUT IN SOFT CLAY IN OSLO

FIGURE 10



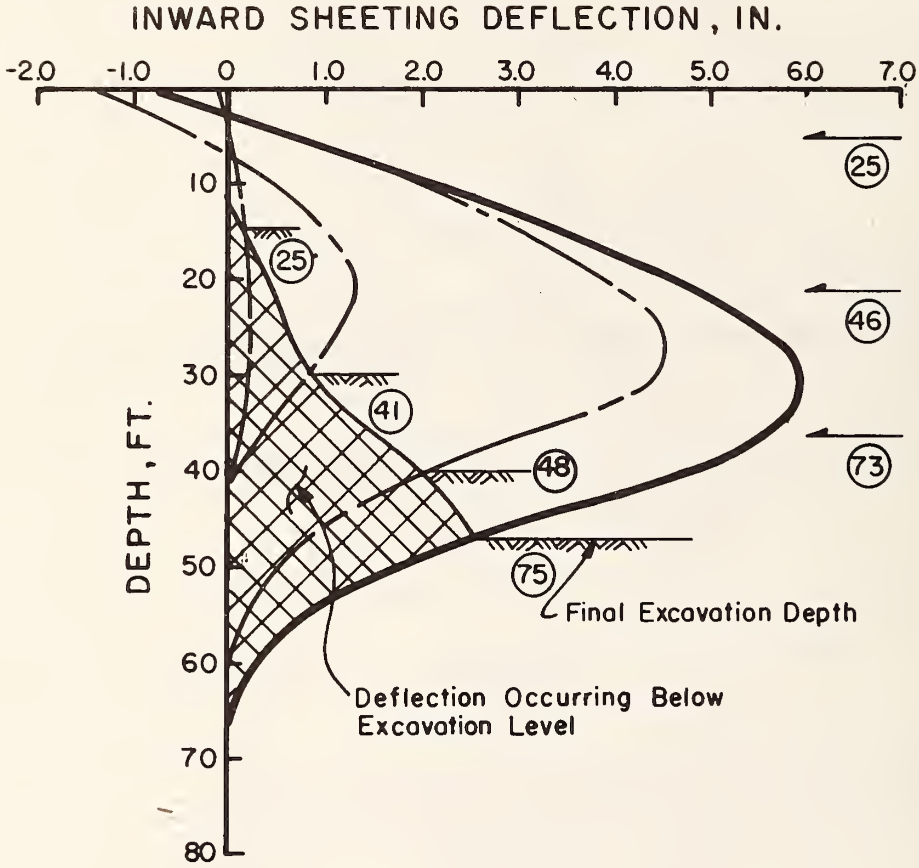
Note: Intially there is no flow and ultimately there is steady-state seepage.

from Lambe (1970)

	ELEMENT A	ELEMENT B
Initial (Static) Pore Pressure, U_o	$A_o\bar{A}_o$	$B_o\bar{B}_o$
Pore Pressure at Steady-State Flow, u_{ss}	$A\ \bar{A}_{ss}$	$B_1\bar{B}_{ss}$
Pore Pressure Upon Unloading	Decreases	Decreases
Pore Pressure During Consolidation	Decreases	Increases
Strain Upon Unloading	Vertical Compression	Vertical Extension
Strain During Consolidation	Vertical Compression	Vertical Extension
Undrained Shear Strength During Consolidation	Increases	Decreases

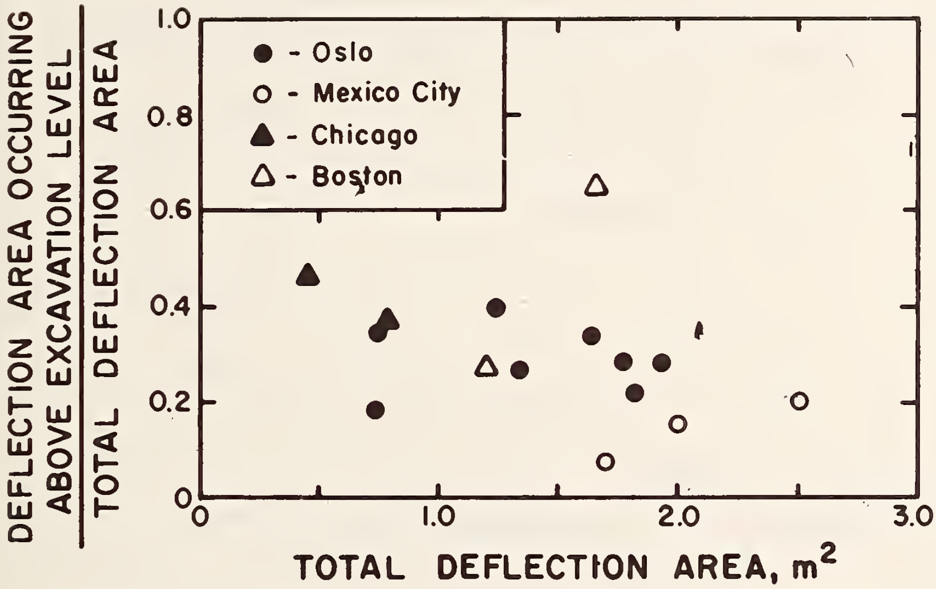
STRESS PATHS FOR SOIL ELEMENTS NEAR AN EXCAVATION IN CLAY

FIGURE 11



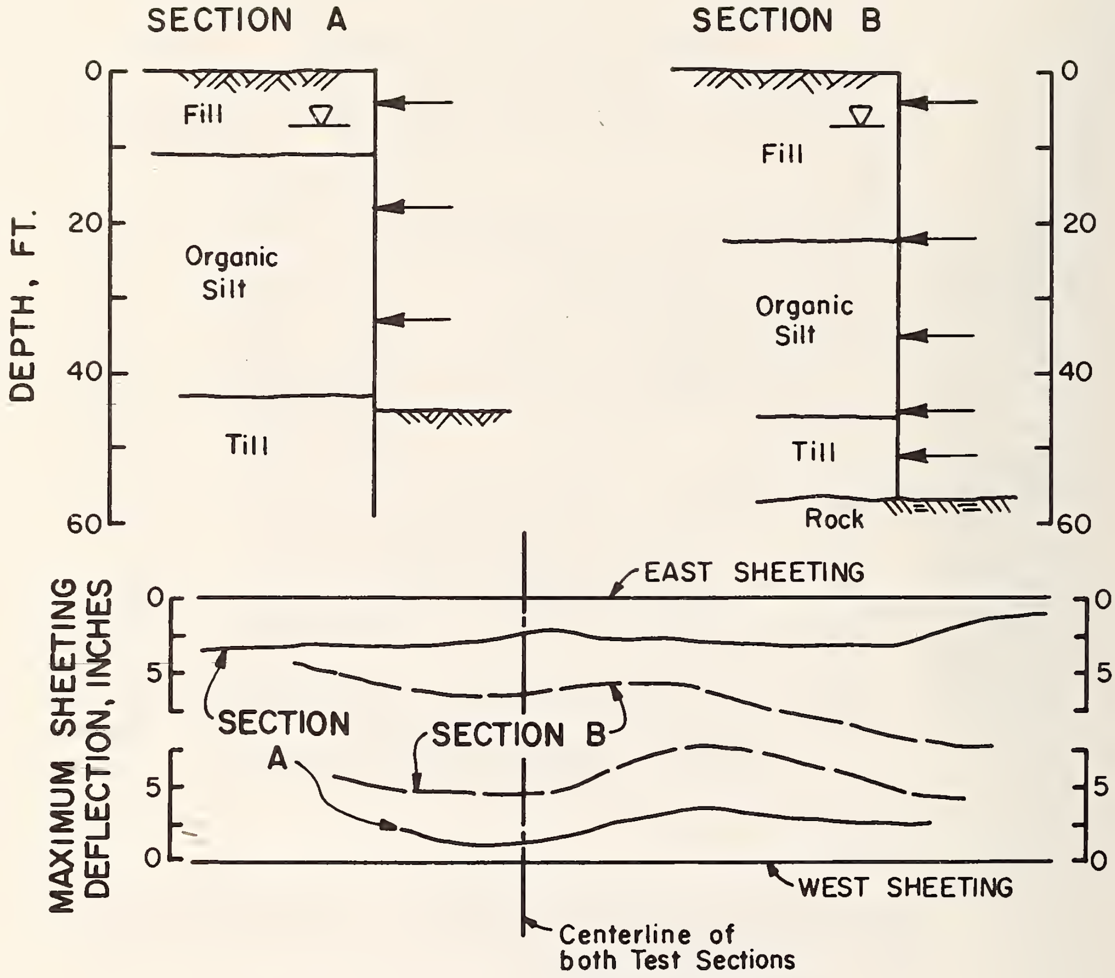
SHEETING MOVEMENTS OCCURING ABOVE AND BELOW EXCAVATION LEVEL FOR CUT IN MEDIUM CLAY

FIGURE 12



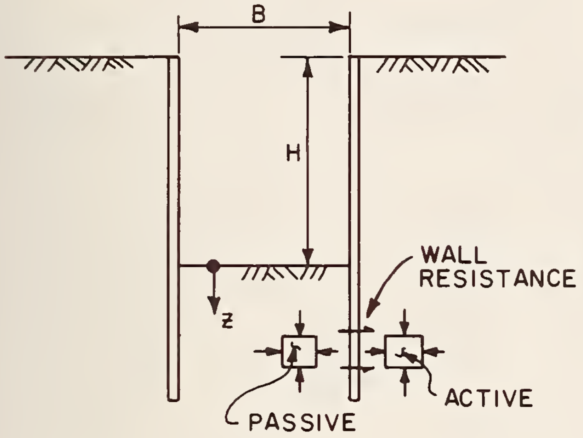
SHEETING DEFLECTION AREAS OCCURING ABOVE AND BELOW EXCAVATION LEVEL FOR CUTS IN SOFT AND MEDIUM CLAY

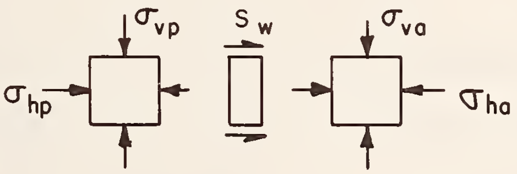
FIGURE 13



COMPARISON OF MAXIMUM SHEETING MOVEMENTS AT TWO TEST SECTIONS OF EXCAVATION IN ORGANIC SILT

FIGURE 14



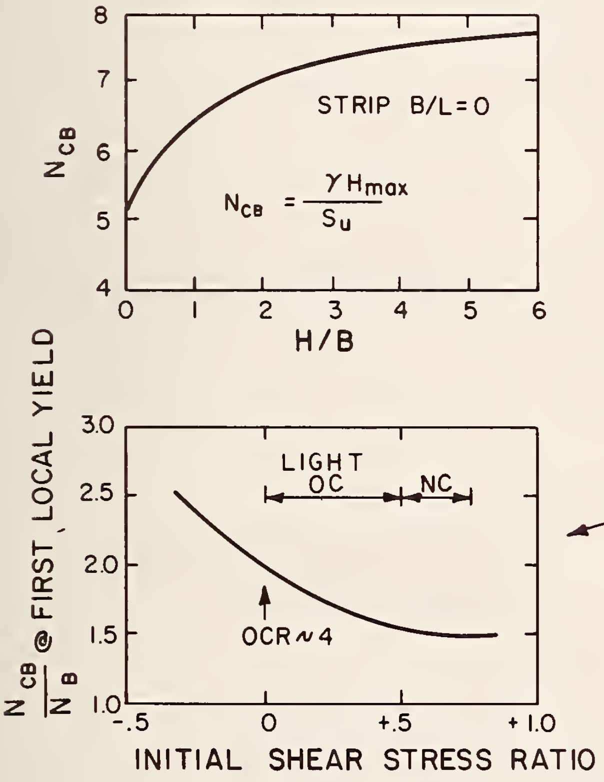


$$\left.\begin{aligned}\sigma_{ha} - S_w - \sigma_{hp} &= 0\\ \sigma_{va} - \sigma_{ha} &= 2S_u\\ \sigma_{hp} - \sigma_{vp} &= 2S_u\end{aligned}\right\}$$

$$S_w = \gamma H - 4S_u$$

$$S_w = 0 \quad \text{FOR} \quad \frac{\gamma H}{S_u} \leq 4$$

$\frac{N_{CB}}{N_B} \simeq 4$ - WALL MUST HAVE STRENGTH TO PREVENT LARGE MOVEMENT BELOW BASE OF CUT

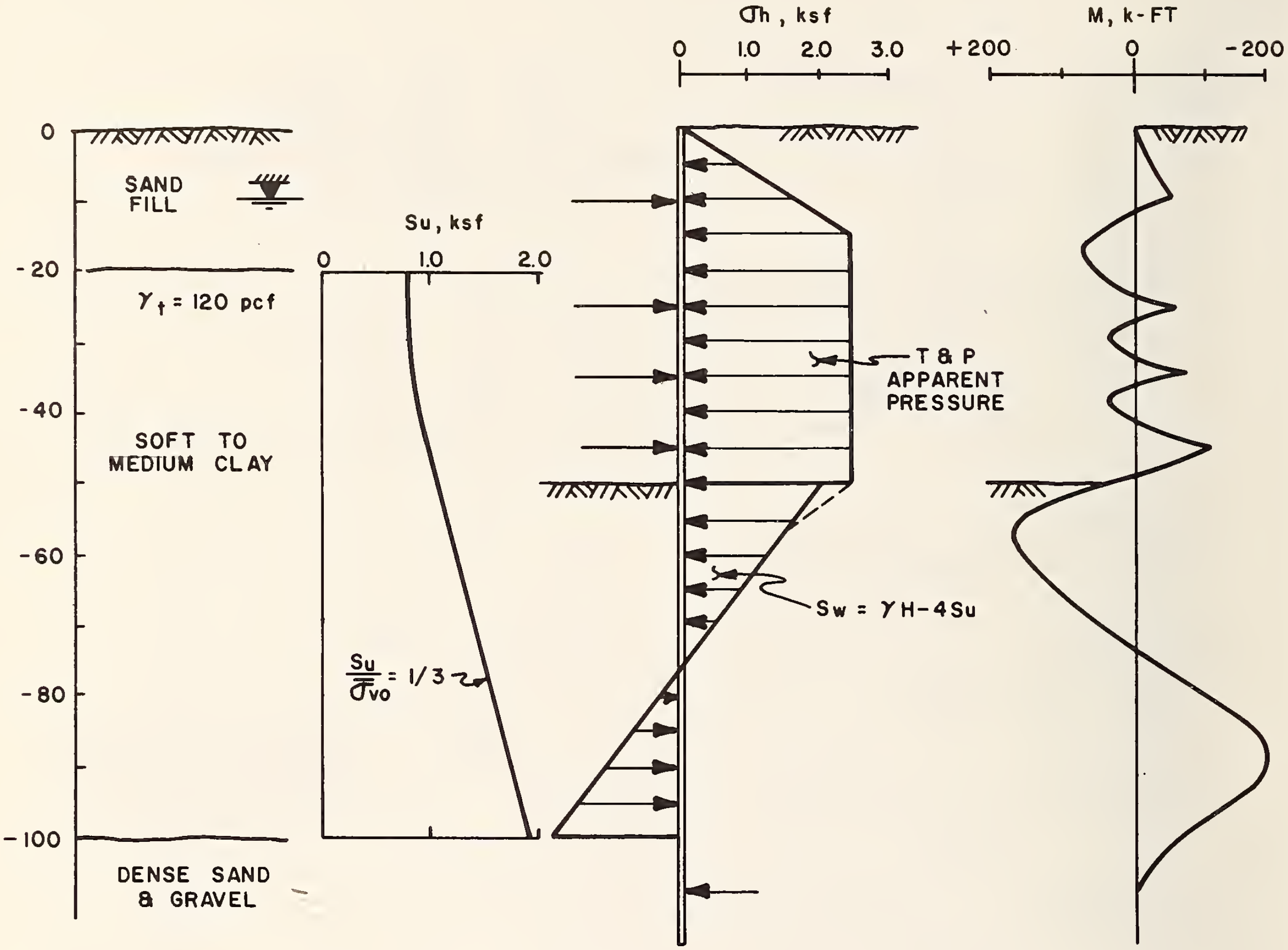


$\frac{N_{CB}}{N_B} \simeq 4$ - FIRST LOCAL PASSIVE YIELD

$f = \sigma_{vo} - \sigma_{ho} / 2S_u$

METHODS FOR COMPUTING FACTOR OF SAFETY AGAINST BOTTOM STABILITY FAILURE AND STRENGTH OF WALL REQUIRED TO PREVENT LARGE MOVEMENTS IN CLAY

FIGURE 15



$N_B = \frac{\gamma H}{Su} \simeq 6 \simeq N_{CB}$

MOVEMENT ~ 1-2 % H

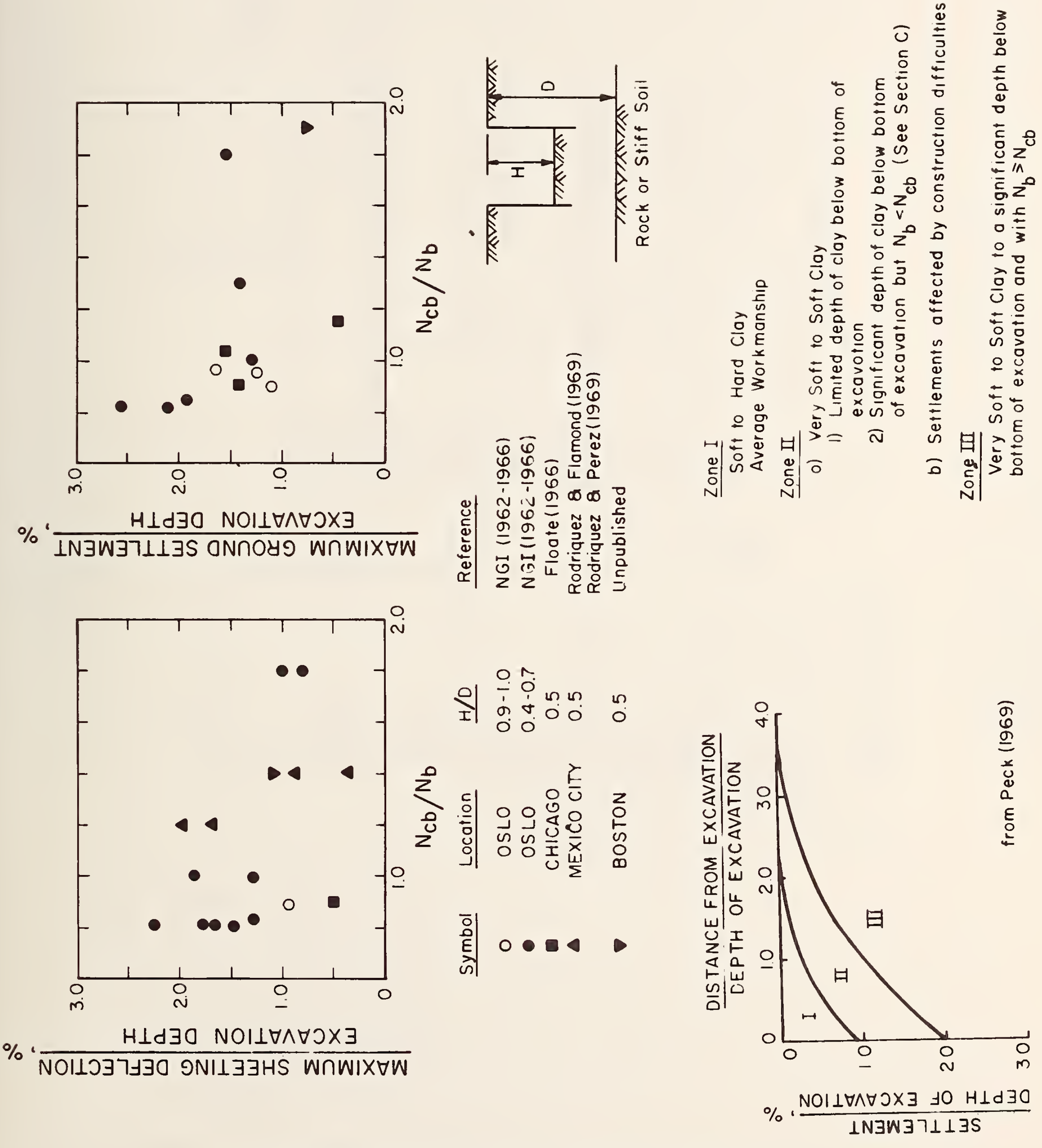
= 6- 12 in.

(FOR CONVENTIONAL SHEETING)

SMALL MOVEMENT (2-3" MAX)
IF WALL PROVIDED WITH ABOVE
BENDING RESISTANCE

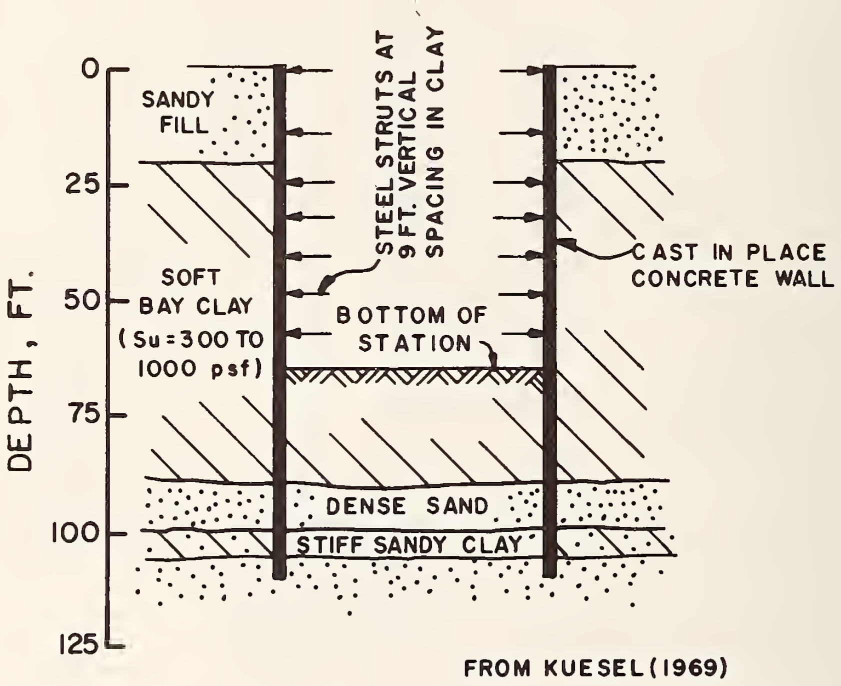
EXAMPLE CALCULATION OF REQUIRED
WALL STRENGTH TO PREVENT LARGE MOVEMENT
FOR DEEP EXCAVATION IN SOFT CLAY

FIGURE 16



MOVEMENTS RELATED TO BOTTOM STABILITY
FOR CUTS IN SOFT AND MEDIUM CLAY

FIGURE 17


DEPTH, FT.
0
25
50
75
100
125
SANDY FILL
SOFT BAY CLAY (Su=300 TO 1000 psf)
STEEL STRUTS AT 9 FT. VERTICAL SPACING IN CLAY
CAST IN PLACE CONCRETE WALL
BOTTOM OF STATION
DENSE SAND
STIFF SANDY CLAY
FROM KUESEL (1969)

CAST-IN-PLACE DIAPHRAM WALL
AT EMBARCADERO STATION IN SAN FRANCISCO

FIGURE 18

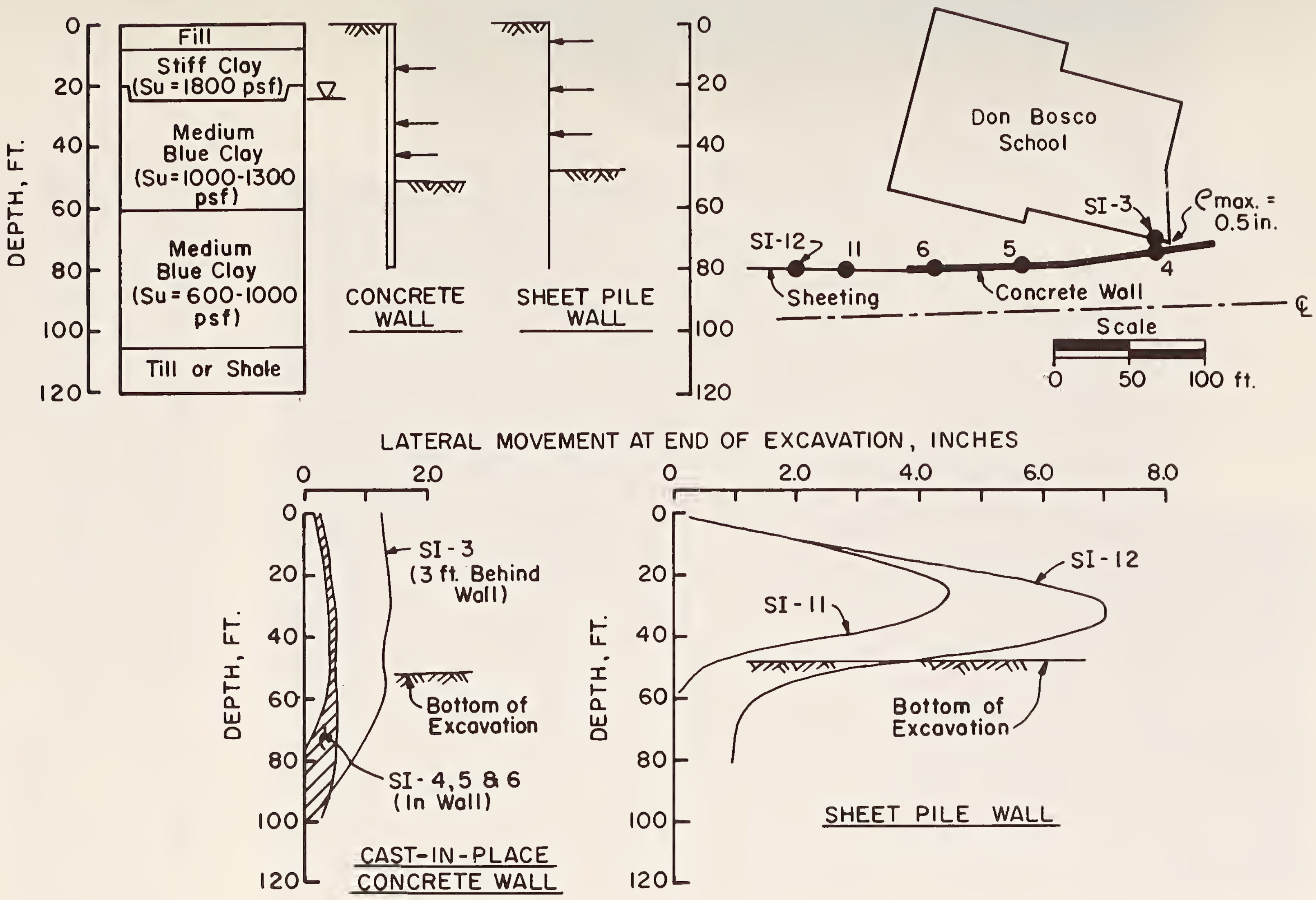
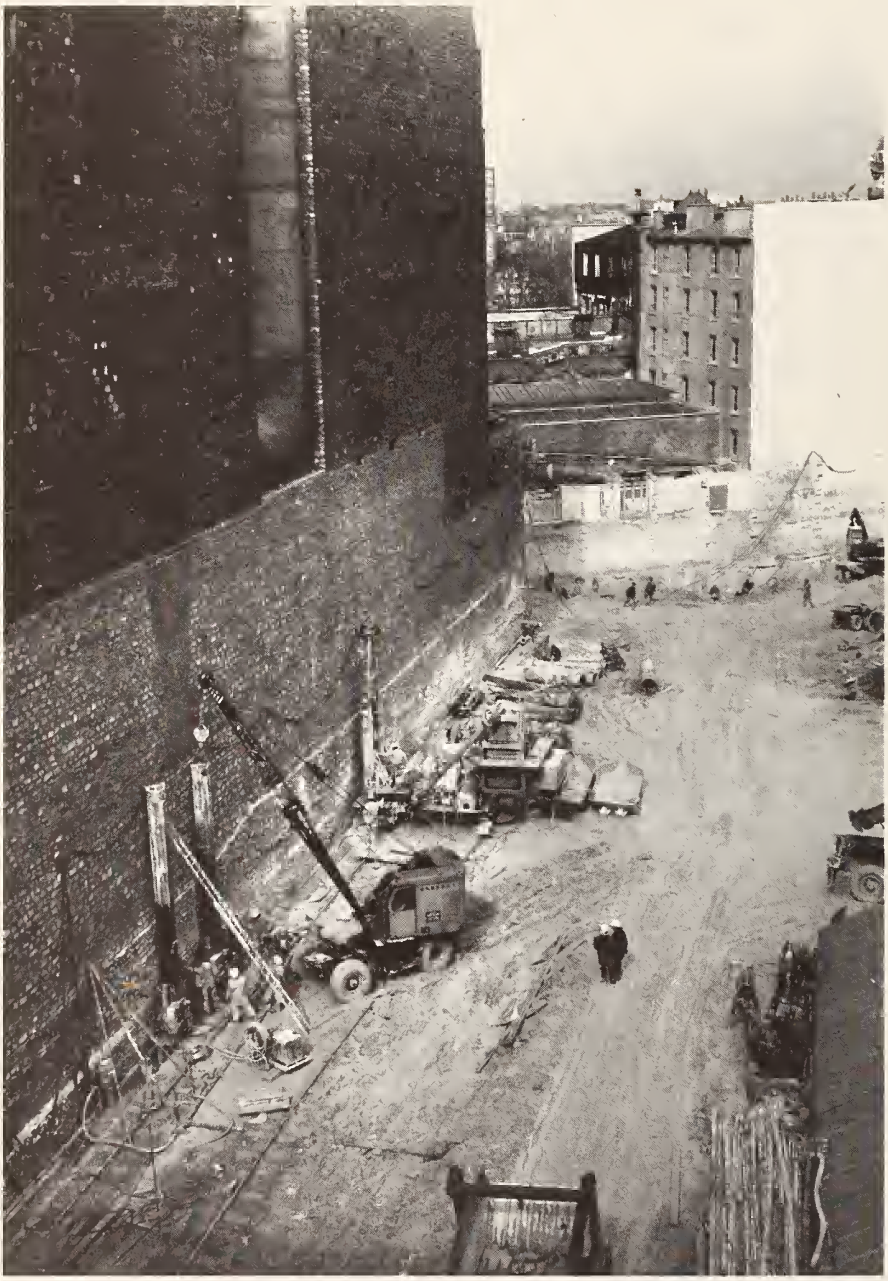


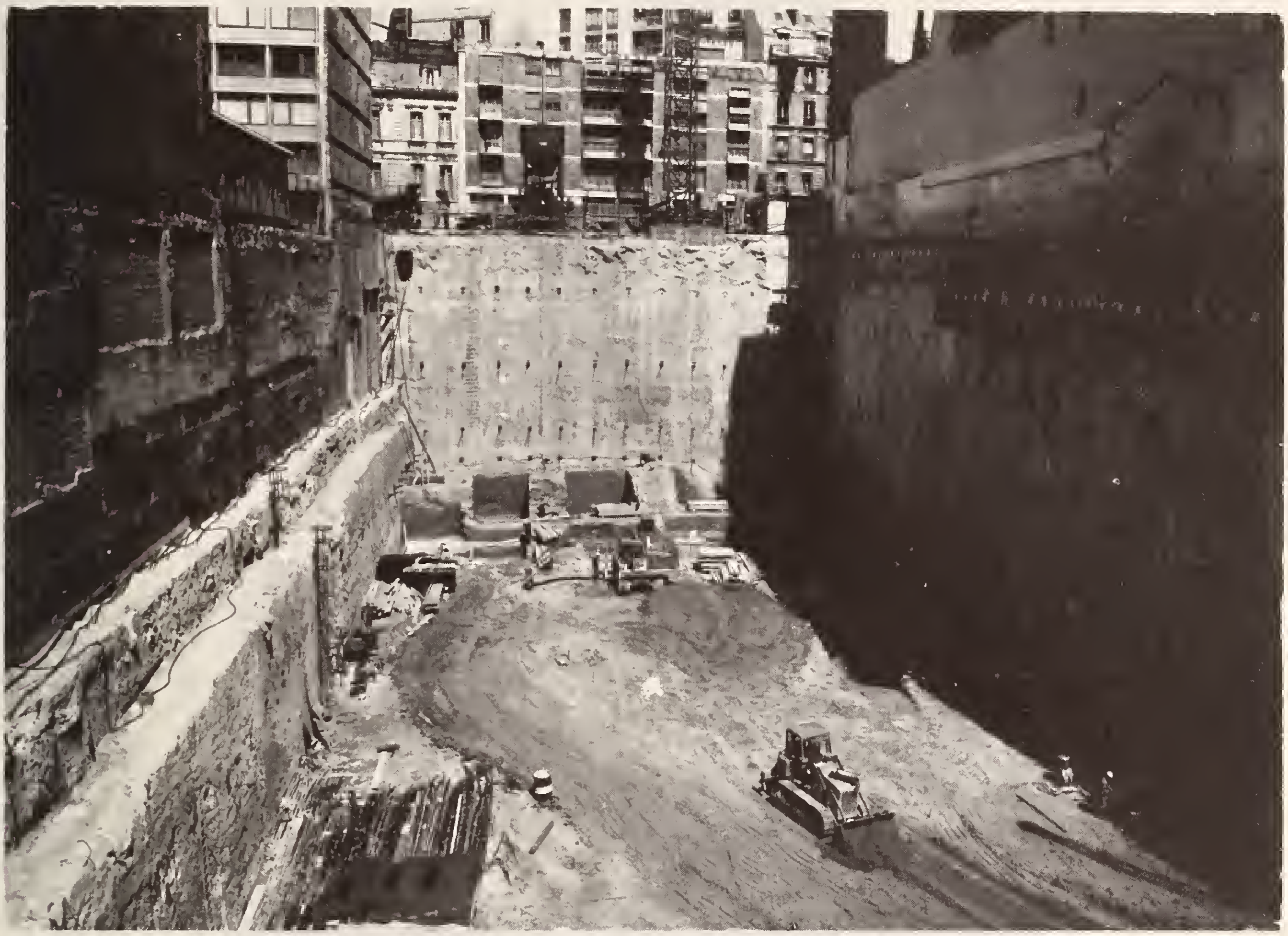
FIGURE 19

COMPARISON OF WALL MOVEMENT FOR CAST IN PLACE DIAPHRAM WALL AND SHEETING AT NEARBY SECTIONS OF EXCAVATION IN MEDIUM CLAY.



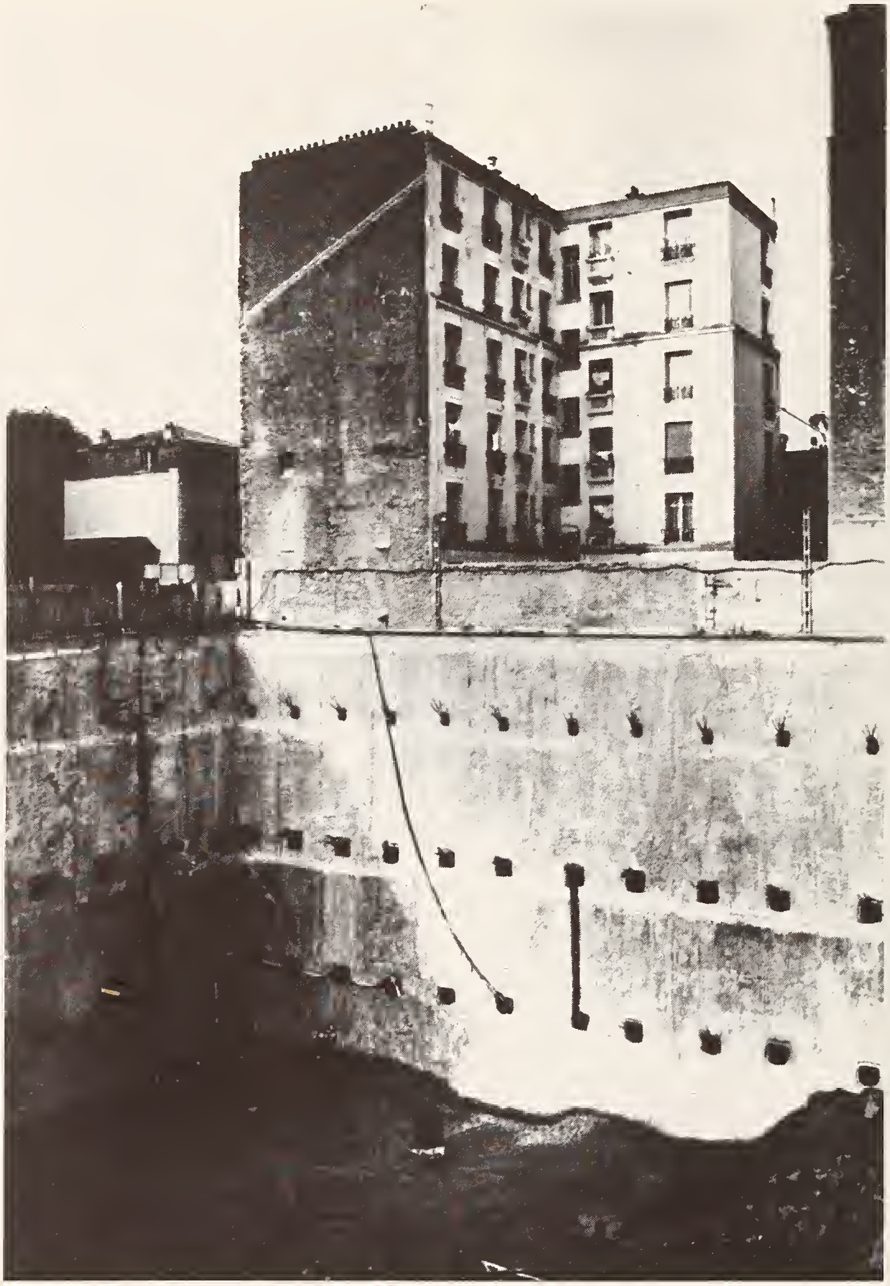
DIAPHRAGM WALL UNDER CONSTRUCTION IMMEDIATELY ADJACENT TO OLD MASONRY BUILDING. PANEL IS BEING EXCAVATED WITH REVERSE CIRCULATION DRILLING EQUIPMENT IN BACKGROUND AND STOP-END TUBE ARE BEING REMOVED FROM RECENTLY CONCRETED PANEL IN FOREGROUND.

FIGURE 20



CAST-IN-PLACE DIAPHRAGM WALL SUPPORTED BY TIE BCKS IMMEDIATELY ADJACENT TO MASONRY STRUCTURES ON SHALLOW FOUNDATIONS

FIGURE 21



CAST-IN-PLACE DIAPHRAGM WALL
CONSTRUCTED IMMEDIATELY ADJACENT
OLD MASONRY STRUCTURE

FIGURE 22

III. CONTRACTING FOR UNDERGROUND SYSTEMS

INTRODUCTION

Successful completion of underground projects requires a close interplay and integration of work performed by several different parties, notably that of the owner, the planner, the designer and the contractor. Indeed other groups such as local, state and federal government agencies; community groups; real estate, finance and insurance companies; interveners and the public are involved and/or intervene in one manner or another in underground engineering and construction. This discussion deals solely with one aspect of the whole problem -- that of contracting for engineering and construction services by an owner with an engineer and a contractor. I stress and urge herein the interweaving of the activities of the engineer and constructor to form a design-construct team as one of the better means of providing an owner with a completed facility on schedule and within budget. Further emphasis is placed on the use of incentive contracts based on time and materials and a variable fee.

The interaction between soil and structure automatically precludes the current adversary role between engineering and construction if disputes and claims arising from unanticipated subsurface conditions, alleged damage of adjacent structures and facilities; relocation of utilities; disruption of streets and loss of business; delays, because of complexity of construction in highly urbanized areas; and similar alleged causes are to be minimized or eliminated.

To obtain more miles of metro system per dollar of expenditure, innovation is essential. Innovation cannot flourish in an adversary atmosphere nor in one where there are disputes of vested interest between owner, engineer and contractor. Innovation in underground construction techniques and equipment results after years of research, experience and investment, and obviously the party providing an innovative technique for underground construction is entitled to a return on his efforts.

An adversary-type contract for underground construction discourages innovation, as neither the engineer nor the contractor wishes to take a risk without knowing the effectiveness and the quality of the work that can be performed with an innovative technique.

RISKS

The design-construct approach -- when work is performed by a team involving the designer and constructor with vested interests involving responsibility, knowledge and experience as well as investment -- assures the owner of the appropriateness and effectiveness of an innovative technique whether it be with a tunneling machine, shield for excavation of soft ground, grouting to precondition granular soils below groundwater, cast-in-situ or precast diaphragm walls to serve both as temporary and permanent support, cutoff walls to reduce dewatering, underpinning, rock bolting, shotcreting and numerous other techniques that engineers, contractors and manufacturers have developed for underground construction projects.

Indeed, underground construction is being publicized today because of the ever-increasing need to preserve and improve the environment in which we live, especially in or near metropolitan areas. Subways, power plants, sewer facilities, utilidors, etc., all require greater use of underground space. Currently, depending on the location and subsurface conditions, it costs between $10 million and $40 million per mile for subway construction.

Despite this high cost, it can be justified for many projects when the trade-offs in land value, cost of operation of the system, city development and similar factors are properly taken into account. Lowering the cost of underground construction will simply save millions of dollars for future planned projects. It has been demonstrated in many cases with recent advances in tunnel boring machines, improvement in the use of conventional explosives, slurry walls, tie-backs and similar innovative techniques that most excavation could be done faster and cheaper. There is resistance to these advances, at least in underground construction, because engineers and constructors are more prone to base the results of their work on precedent rather than on innovation. This is certainly prudent, provided innovation is not neglected. When one investigates the reasoning behind this reluctance, risk is found to be the basic issue. The risks stemming from uncertainties in underground construction are not defined nor clearly assigned in the contract documents; and when disputes arise because of the consequences of these risks, there are claims and counterclaims and long and involved arbitration and litigation on resolution of the disputes which should not have occurred in the first place had they been properly defined and assigned to the three parties involved -- owner, engineer and contractor.

There are risks that belong to the owner. It is his facility and site, and certainly there are risks in which he should share. There are risks of error and omission that belong to the engineer stemming from the subsurface investigation, field and laboratory testing and the interpretation of data relative to the design of the facility. There are obviously risks that belong to the contractor involving the effectiveness of the techniques that he plans to use for construction, the influence of the construction techniques on the facility itself, construction risks relative to safety such as cave-ins and similar risks involving construction procedures planned in accordance with the subsurface data presented in the contract documents but which construction discloses are different. Unfortunately, data obtained for the design of the facility may be neither adequate nor sufficient for the contractor for his interpretation relative to the proposed construction technique or procedure that he plans to follow in the installation of the facility. Further, there are risks involved with unanticipated subsurface conditions, movement of adjacent utilities and buildings, strikes, poor weather, and delays in delivery of materials. These risks do not necessarily belong solely to the contractor. In many cases, these should be assigned, as well as their consequences, to the owner.

To avoid the disputes and claims, the risks and uncertainties involved with engineering and construction, it appears that these can be best handled through a design-construct approach. Each expected or unexpected change of condition in the subsurface, site or otherwise, can be taken up with the owner and his manager during the course of the design, and appropriate agreements negotiated prior to construction of how such uncertainties and their consequences would be treated during the course of construction of the project.

To illustrate this point, consider how the risk of lateral movement of a retaining wall for a cut-and-cover operation can be minimized. Typically, soldier piles may be driven and lagging installed with internal bracing to provide lateral support of adjacent streets and buildings. There is movement of the street during excavation and the installation of horizontal braces because of the flexibility of the wall. There is further movement or settlement of adjacent utilities and buildings if dewatering of the excavation is needed. During construction of the tube and removal of the horizontal bracing system and the placing of backfill between the permanent and temporary walls, there is further movement and subsidence of adjacent facilities. Finally, there is the extraction of the piles with some further disruption and, in certain cases, movement.

In lieu of this more or less standard operation, consider the construction of a cast-in-situ or precast diaphragm wall which can serve as both a temporary and permanent wall, using a bentonite-cement-slurry trenching technique. During excavation and the installation of the horizontal bracing system, the lateral movement is minimized because of the small disturbance of the ground and the rigidity of the concrete wall. Upon completion of the excavation and the placement of the invert and the roof of the subway, there is no disruption from backfilling insofar as adjacent utilities and buildings are concerned. There might be some settlement of the street, depending on the backfilling, but this condition would be common to both procedures described above.

The diaphragm wall technique has been used for decades in Europe and is common in the construction of metros and large openings for building foundations. Diaphragm walls have few precedents in this country.

With the standard bracing system, in many cases the details of the bracing method of installation, etc., are fully given in the contract documents. The contractor has little leeway to change the support system and is held responsible for its effectiveness. If observed movements are greater than predicted, the contractor has to change his operations and quite often without compensation. Delays may occur in the construction work until a new support system is designed and, again, often without compensation to the contractor. Ultimately, there are disputes and claims.

On the other hand, with a design-construct approach, the responsibility of the design and installation of the bracing system rests with the engineer-constructor. Different construction techniques for lateral support are evaluated and the effects of the construction operations are integrated into predictions of lateral movement. An innovative, proven, practical support system such as a rigid diaphragm wall can be readily adopted and modified to actual site conditions without resulting in disputes over division of responsibility and effectiveness of both the design of the wall and its construction.

Another factor often not properly considered is the relative cost of the two systems. A diaphragm wall may cost five to ten times more than the temporary wall; but considering trade-offs and especially that the diaphragm is both a temporary and a permanent wall and that the rigidity of the diaphragm wall practically precludes damage to adjacent utilities and buildings, certainly the use of such innovative techniques has a place in the engineering and construction of underground facilities.

It must also be recognized if engineers and contractors are to assess properly innovative techniques, reduce risks, define and assign risks, eliminate adversary roles, improve quality of construction and increase the purchasing power of each dollar spent for a subway system, that the procurement policies of the various state, local and federal government agencies, as well as authorities and other owners of underground facilities, have to change from our current practice of divorcing engineering from construction.

GEOTECHNICAL CONSIDERATIONS

Geotechnical considerations play an important role in the planning, design and construction of underground tubes and stations for urban mass transportation systems. It is essential that subsurface conditions be explored and tested for both design and construction. Much of the work that is done for design is also useful in planning and carrying out construction operations, but subsurface data gathered for design are not necessarily sufficient and adequate for construction. It is important to recognize the interplay between design and construction. The interplay is a function of construction techniques and procedures that are to be followed in the building of a facility. This indeed makes it practically mandatory that a contractor be involved in the planning and design of the facility almost from its inception if the owner is to obtain the completed facility within the shortest period of time and at least cost.

Major planning decisions involving the alignment and grade of a subway and the type of construction to be used whether it be for a tunnel or a cut-and-cover are strongly influenced by the type and design of the structural support system, the requirements for protecting adjacent utilities and buildings, the methods used for construction and, in many instances, the format of construction specifications and contract documents. In other words, the feasibility and cost of the underground system are controlled to a significant extent by the character of the subsurface materials and the construction procedures.

The geotechnical engineering problems associated with underground construction are complex due to the influence of many interrelated technical factors and because of the influence of the difficult-to-control contractor behavior. Moreover, the subsurface conditions in many metropolitan areas are poor. Most major cities are near navigable waterways where the underlying soils are often weak and compressible and the groundwater level is near the surface.

The interrelationships between engineering and construction and the consequent need for an integrated approach to the engineering and construction of subways should be understood by the owner, and he should give serious consideration to the adoption of a design-construct approach in his contract for the execution of the work.

Potential damage to adjacent streets, utilities and buildings caused by movements associated with tunnel or cut-and-cover construction is often a most important design criterion in selecting the alignment and grade of the subway and the type and method of its construction. Therefore, particular emphasis needs to be given in predicting movements and their effects on adjacent structures. This can be accomplished best through a system that interweaves engineering and construction.

CONTRACTING BREAKTHROUGHS

Lowering the cost of underground construction will save millions of construction dollars for planned projects. This can be accomplished by improvements and innovation in the development of new and better technology, especially in the application of this technology to underground construction projects.

A cursory review of current literature and publications concerning underground engineering and construction indicates that rarely are underground projects involving tunneling or cut-and-cover completed on time and within budget. It has been stated that approximately 75% of such projects have overruns averaging at least 30% of the bid cost and overruns of 150% or more are not uncommon. Reasons for the delays and cost overruns are as numerous as the projects themselves and range from unexpected labor costs to inadequate surface data, disputes in the interpretation of contract documents, delays in delivery of materials, breakdown and inappropriate equipment for the construction operations, and numerous other reasons.

Individuals who are in the position of paying for underground construction facilities are keenly interested in achieving some form of a breakthrough that will lower the cost of such projects and eliminate lengthy construction delays. Major advances in technology or breakthroughs are seldom planned or scheduled. Generally they happen when the right conditions exist. For example when (1) an evaluation is in progress; (2) there are real and urgent needs; (3) innovation is permitted; and (4) a basis for reward or incentives exists. While it is clear that advances in underground construction technology are both required and, in fact, sought, it is equally unclear as to where a true advance will come from. This is not to suggest that on-going research and development projects are totally lacking. Nothing can be further from the truth. Government agencies at various levels carry out in-house and outside research in universities and industry. Material and equipment manufacturers are continually modifying and improving their products. Unfortunately, much of the effort for technological development in underground construction is fragmented and disjointed. The pieces simply are not being integrated. Also coupled with this fact is the hesitancy to use new products, methods and systems. Distrust for the new or innovative approach is perhaps stronger in underground construction than in any other branch of the construction industry, because of the risk involved when unanticipated subsurface conditions occur. The contractor with an innovative technique, lacking experience, simply does not want to chance the losses.

Considering the disputes and alleged claims and the ever-rising cost of underground construction, it appears reasonable that a major breakthrough can be achieved in the economies of underground construction by simply altering and reevaluating our current contracting procedures. Currently used, advertised contracts based on lump sum have such serious shortcomings in underground construction that both the engineering profession and the construction industry are today seriously studying, along with government agencies and individuals from private industry, contracting procedures that should be adopted to minimize and hopefully eliminate the adversary roles that exist in construction.

There is no one simple form of contract that is best for all projects. However, there appears to be, in the opinion of many, a form of approach to construction that has merit and this is the design-construct approach which can be carried out under different forms of contract--whether they be lump-sum, unit price with a fixed fee, cost-plus, time and materials, incentive fee, etc.

Recently, the Bureau of Reclamation had planned a major field research program to include the contractors, equipment manufacturers, engineers and geologists from which one of the major objectives was to evaluate the effectiveness and merit of a cost-plus incentive-fee-type contract which had built into it a major portion of the work to be carried out on a design-construct basis. The cost-plus-incentive-fee contract proposed by the Bureau of Reclamation was to provide the contractor with an incentive to advance the program of construction as rapidly as possible. When the work was completed, the fee would be adjusted upward if the cost was less than the target cost, and the fee would be adjusted downward if the cost had overrun the target cost. There are those who believe that a major contribution and breakthrough in underground construction will come from writing and using this type of contractual procedure. Experience has shown that a fair and creative set of contract documents can greatly enhance the contractor-engineer-owner relationship. The result would be a more efficient and economically executed project.

At a meeting of the Excavation Technology Discussion Group in April 1972 under the auspices of the National Research Council, one of the recommendations which was given "most urgent priority" was "to develop contracting systems that would permit and assure engineered-construction." Such contracts are not new to the construction industry. The design-construct approach has worked in such areas as multi-family housing and process plant construction. Properly controlled, such a contractual system could work for underground construction.

The Underground Construction Research Council, founded and supported by the American Society of Civil Engineers and the American Institute of Mining Engineers, has published a report which summarizes work done under the sponsorship of the National Science Foundation. The authors of this report strongly recommend a detailed study of contractual arrangements which could permit combining the experience of the engineer, geologist, equipment manufacturer and contractor. The report states that a 30% cost reduction might be possible with a creative arrangement using people, hardware, formats and procedures which are available today. Recently, the U. S. National Committee on Tunneling Technology has undertaken a major review of contracting procedures. This study is supported by a grant from the National Science Foundation.

OVERRUNS

Overruns in underground construction stem from many factors. To mention a few:

1. Site and subsurface conditions different from those anticipated;
2. Unexpected damage and claims from movement of supporting soil;
3. Claims stemming from specifications and drawings;
4. Inappropriate parcelling of the project to subcontractors;
5. Inadequate consideration and evaluation of the construction procedures on the predicted behavior of soil, rock, support systems, etc., during and following construction;
6. Risks inappropriately assigned and not in keeping with the intent of the contract or understanding of the contractor.

To reduce overruns and the alleged claims from additional work requires a system which:

1. Allows for innovation and provides incentives for this innovation:
   (a) for the owner to achieve a satisfactorily performing facility within cost and schedule;
   (b) for the engineer to use best available techniques for exploration, investigation, analysis, design, prediction, etc.
   (c) for the contractor to minimize delays, improve project management and increase profits.
2. Shares risks appropriately in accordance with the work being performed. Excavating a tunnel or opening to receive liners

or providing a braced wall has risks much greater than the actual installation of the liners, precast tunnel sections, and mechanical equipment. This implies:

(a) the assigning of larger risks to soil conditions than construction of the liners and mechanical equipment, per se;

(b) the evaluating of risks associated with labor disputes, strikes, delivery of materials and weather on a basis different from the risks involved with uncertainties of subsurface conditions;

(c) the allocating of contingencies to the contractor for those risks where he has control of the construction procedure, flow of materials, etc.; while risks that cannot be controlled by the contractor -- for example, the unexpected soil conditions -- should be borne by the owner. The contract agreement should be adjusted accordingly and the risks should be clearly defined and assigned. The contractor should not carry contingencies in his costs for risks that are to be borne by the owner.

(d) the assigning of risks as to the adequacy, reliability and sufficiency of subsurface data to the engineer within reasonable limits. Normally, an expenditure of the order of 1/10 of 1% of the total cost for an underground facility is spent on the investigation of the subsurface conditions. This expenditure may rise to as much as 1%. depending on the amount of drilling, field testing, geophysical work, and consulting services required for interpretation of the data. To reduce the risk of unexpected subsurface conditions, the investigative effort should be increased consistent with the appropriate return therefrom. Upon completion of reasonable, adequate subsurface exploration for the engineer and the contractor, those unanticipated subsurface conditions that are exposed during construction and the consequences of these unanticipated conditions, especially as to cost overruns, delays and/or changes in design, should be borne by the owner.

3. Considers the influence of the construction procedure on the predicted performance of the facility. This implies instrumentation to observe movements, loads and altering construction procedures if distress might result as a consequence of the construction procedures both during and following completion of the facility.
4. Allows for flexibility, direct communication, and interaction between the various parties involved with the planning, design, construction and operation of the facility.

The key to the development of such a system is implementation of proven innovative construction techniques that would have an impact on the underground construction industry and procurement policies compatible with this. Implementation not properly timed, relative to need, could mean excessive and unnecessary overruns in failure to meet an objective of fully utilizing appropriations for subway facilities.

CONTRACT MODEL

Background

Among many choices, the design-construct or engineered-construction approach with incentives appears to offer for underground works the best promise for a "contractual breakthrough." Figure 1 illustrates the design-construct approach relative to the current adversary approach for underground construction. In the adversary approach, the owner initially retains an engineer to perform the necessary investigations, design the structure and prepare the contract documents for bidding. The engineer is usually paid for services on a fee basis or on the basis of a cost incurred plus a reasonable profit. Following bid receipt, a contractor is selected generally on the basis of low bid price and qualifications. A performance bond is obtained to assure that the project will be finished by the contractor or his bonding company at the bid price plus any extras claimed by the contractor and validated by the owner.

With a design-construct approach and incentive contract or some equivalent payment basis, the owner selects a project manager from his own staff or retains a project manager from a firm that has background and experience with both engineering and construction. The project manager controls the project for the owner from its inception through the various stages of planning, financing, design and construction. After planning, to establish feasibility and function and arrangements for financing have been made, the project manager (should he elect to follow a design-construct approach) selects an engineering-construction firm or an engineer and a contractor to work as a team in performing the initial investigations, design and construction of the facility. The engineer inputs the necessary technical knowledge and evaluates the design parameters while the contractor assures that decisions made during the design phase are conducive to the lowest cost and that the construction can be performed on schedule, using techniques and procedures most compatible for the particular project. The contractor selected should be experienced with the type of planned project in order that he evaluate significant design factors which could adversely affect the construction and cost of the facility. The engineer-contractor team completes the design and prepares the construction documents. These documents can be typical contract documents for bidding or only sufficient drawings and technical specifications to detail the proper construction of the facility.

If construction proceeds on a time-and-materials basis or a unit-price basis under the control of the project manager, the construction documents need not contain long, complex and often confusing legal descriptions of measurement and payment. Under this arrangement, construction begins at the earliest possible date without delays for bid evaluation or contract negotiation. Construction materials requiring long lead times can be ordered immediately. Preliminary site work can be started and other aspects of the project construction entered into which, under the adversary construction approach, would be delayed until after the contract award.

A design-construct approach can also be used to advantage when bidding is required on a lump-sum basis. Following preparation of contract documents, the project can be let out for bids with the engineer-contractor team being allowed to bid on the project. Although other contractors may submit qualified bids, the owner can be virtually assured that the contractor who has worked with the engineer from the initial phases of the project will submit a bid within the budget. This team is thoroughly familiar with the project and the various cost items involved. Unreasonable contingencies need not be included in its bid price to account for unknowns. Unanticipated subsurface and site conditions, the consequences of inadequate or incomplete subsurface data, or other aspects of the construction which, under normal bid situations, a contractor considers in his lump-sum bid would, of course, also be included in the design-construct team's contingencies.

Perhaps the question of whether the engineer-contractor should be permitted to bid or not has not been adequately considered. It may not be immediately clear when contract documents are prepared by such a team and a lump-sum contract is to be awarded, whether an engineer-contractor team should be permitted to submit a bid or be excluded from a bid. There are advantages and disadvantages with either choice. Questions of vested interest, points of conflict, quality assurance and economy can be raised with both cases. Obviously, further consideration and evaluation of actual experience where contracts have been let to a design-construct team to execute the construction on a lump-sum basis should be compared with similar projects where the team provides only management and quality assurance and construction is performed by a general contractor and his subcontractors.

Payment for engineering services rendered under a design-construct contract could be similar to that made with current contracts where engineering and construction are separated. If construction proceeds with the selected contractor's forces, the engineer-contractor team is paid a fee and, if incentives are included, a variable fee based on the difference between the target estimate and the actual cost of the project can be adopted.

Under this approach, bonding is not necessary since the owner has retained and is compensating a qualified, selected team to control the schedule and cost of construction. If the construction team were to have financial difficulties and leave the project without completing the structure, the project manager selects another contractor to continue this work on a time-and-materials basis. Delays due to negotiations with bonding companies, rebidding or resolution of claims, etc., are not required. Initial proper selection of a competent and qualified contractor as the member of the design-construct team should, however, preclude this occurrence.

ADVANTAGES AND DISADVANTAGES OF THE STANDARD LUMP-SUM ADVERSARY APPROACH AND SUGGESTED DESIGN-CONSTRUCT INCENTIVE APPROACH

In the adversary system, the firm bid price submitted by the contractor will include contingencies for unknown conditions and/or circumstances which the contractor feels may arise during construction, as well as conditions that may be imposed by the engineer, inspector and others relative to the interpretation of the drawings and responsibility for subsurface conditions actually exposed during construction. If such conditions do arise, the contractor utilizes this contingency and the owner pays for work accomplished. On the other hand, if the unknown conditions do not occur, the owner still pays the full bid price and the cost to him is higher than the actual construction cost.

Also, if conditions or circumstances become significantly different than anticipated by the contractor, he will claim extras and additional compensation and the owner will still pay at least the cost of construction and, in many cases, even much more should the claims involve long delays which occur often and in added costs associated with arbitration or litigation. If conditions of construction change in favor of the owner, some credit can be requested from the contractor, but on practically all projects the credit items are much less than the extra items. This potential of claims from the contractor often limits the flexibility of the engineer during construction because logical modifications to improve quality or save money are difficult to implement since negotiation with the contractor to modify compensation is required.

The amount of contingency that a contractor includes in his bid is dependent on (1) his experience on similar projects, (2) his present workload, (3) his familiarity with the project, (4) how definable the parameters of construction are, (5) the uncertainties and risks and the manner in which these have been defined and assigned in the contract documents, and (6) the degree of risk the contractor is willing to accept. Considering all types of engineering structures, the greatest amount of contingency will typically be included on bids to perform construction for underground facilities and earthworks since field conditions for this work (as opposed to above-ground structures) can be more variable and because conditions are less understood by the contractor.

Only a contractor who is fully experienced in the engineering aspects of a facility will submit a bid with prudent and reasonable contingencies. If a prudent and reasonable bid can be obtained from such a contractor, the adversary system may, on certain projects, be to the owner's advantage since he can be assured of a bonded fixed price and can obtain the facility at that price plus extras for actual change of conditions agreed to during construction.

With an incentive contract and especially one using a design-construct approach and construction performed on a time-and-materials and/or unit-price basis, the owner pays for the actual cost of construction, including only those contingencies which actually occur during construction. If modifications to the design and field changes are advisable or desirable, the owner pays only the actual increased cost or receives the full benefit if modifications result in decreased cost. Modifications can be made quickly and easily by field decisions rather than lengthy change order negotiations and contract revisions. With the design-construct approach and an incentive contract based on time and materials and/or unit prices, the owner does, however, accept the risk of costs higher than would have occurred under a lum-sum adversary approach if (1) both engineer and contractor have misjudged the cost including contingencies where, in a fixed price, the contractor would be forced to take a loss on his particular project; or (2) inefficiencies in construction techniques or procedures cause the cost to rise above the target price. It is then prudent for the owner to select a design-construct team who is experienced and knowledgeable in the engineering and construction aspects of the project to assure that inefficiencies and cost waste do not occur.

In many instances, the design-construct approach (with an incentive contract) has cost schedule and quality advantages over the adversary approach; but the owner must be willing to accept risk and must rely on his project manager working in cooperation with the selected design-construct team to provide sound and prudent management; competent and practical technical guidance and design; adequate and reliable geotechnical input and knowledge and experience with different construction techniques and procedures; and to control effectively and efficiently the costs, schedule and quality of the work. The design-construct approach with an incentive contract has been used on projects in recent years and has been successful. The advantages and gains in both cost and schedule repeatedly prove to outweigh the risks involved in working on this basis.

CONTROLS REQUIRED

When a novel or innovative approach is tried, it is highly desirable to utilize sufficient control mechanisms to prevent a total disaster from occurring either technically, financially, or otherwise. Control of underground projects is indeed complex, primarily because adverse geology can present circumstances that are difficult to deal with. The variability of the subsurface conditions is a good reason to spend sufficient time and money during the exploration and preliminary design phases to insure that all possible contingencies are given consideration. If a site has been carefully assessed by a competent, capable engineering geologist who has had the opportunity to perform necessary and sufficient field and laboratory tests, it is possible that potential problems from contingencies can be reduced to a low order of probability. If this can be achieved, control of the project during construction is an easier task. The basic premise is that if contingencies can be reduced in the real sense, then cost estimates and schedules can also be more realistic. Control of the project becomes a problem of controlling contingencies and monitoring costs and schedules. There are countless ways--some very sophisticated--for doing so.

The engineer who selects and designs the foundation system must constantly consider potential problems associated with construction. Many times construction considerations are absolutely crucial to the success of the project. Before selecting the support systems, the designer will generally make either a general or detailed comparison with other alternate support systems with which he has had experience on similar or near-similar projects. Even if the proposed support system and subsurface conditions appear to be favorable for construction, experience has shown that the designer and the contractor can get into more trouble, often difficult to remedy, from unanticipated sources during the boring of the tunnel, excavation of the shaft, dewatering, and excavating and bracing for cut-and-cover projects.

Four major considerations for successful and economical underground engineering and construction are:

1. Assurance that the site and subsurface have been sufficiently investigated for a careful and proper evaluation of the effects of potential construction problems before selection of the construction techniques and procedures;
2. Selection of a contractor with proven capabilities;
3. Insistence that adequate field engineering supervision and control be provided to assess subsurface conditions relative to the predicted performance of the facility;
4. Flexibility on the part of the contractor to adjust construction techniques and procedures to unanticipated subsurface conditions uncovered during construction.

Every effort should be made to convince an owner to award the construction work to a contractor who has good equipment and procedural experience even if the bid given as a unit price or lump sum from an inexperienced contractor is lower. Experience has repeatedly shown that an award to an inexperienced contractor on a low-bid basis is a poor choice because of cost extras claimed at the completion of the work; or, more importantly, because the entire project schedule is delayed due to lack of proper equipment or technique. The more difficult the anticipated project, the more important it is to select a qualified contractor.

Contingency Control

As previously stated, the contingencies of underground construction are difficult to forecast accurately. Currently, the engineering profession and the construction industry has at its disposal virtually countless sophisticated exploration tools along with many systems to analyze the resulting data for the subsurface exploration of sites and preparation of designs. But since the design for many major underground projects is not complete until the project is constructed, the owner must recognize the need to deal with changes during construction. The design-construct approach provides an excellent means of dealing with these changes.

Changes during construction, which occur on most projects, lead to situations where an adverse contingency which becomes a reality during construction is greatly magnified by lack of quick and decisive action. Delays can result from either a misunderstanding of the severity of the problem or lack of contractual set-up that permits efficient implementation of remedial action. There is need to expend an effort in devising a system to deal with contingencies of a project. The design-construct approach offers a way to increase the probability of having a successful project and to insure that design and construction are truly integrated, allowing for prompt field decisions.

The choice of a protection scheme relative to underground construction work focuses attention on the issue of contingencies and risks. At some point in the design-construct sequence, a decision as to the protection scheme of adjacent buildings, utilities, streets, etc., has to be made. Structures needing protection require that engineering focus on developing a low cost but effective and adequate scheme. Many structures adjacent an underground project can be classified as very low risk and need only survey points. A preconstruction survey and observation to check performance are adequate.

A large unknown factor regarding construction costs, however, centers on those intermediate cases where it is not obvious whether a structure requires some form of support such as underpinning or not. If specifications have determined a particular protection scheme for these structures, then presumably the bids would reflect the scope of work. Actual cost, however, may increase since the contractor might be held responsible for any damage regardless of how the project is specified. This item of who is responsible for damage to adjacent structures when the protection scheme is specified, especially with the current adversary approach, is indeed an uncertain element in the construction industry and in its legal interpretation.

There is a need to define uncertainties, risks and to assign these factors to the various parties concerned with the current two-phase approach to a project involving engineering on the one hand and construction on the other. Performance specifications offer at least a reasonable means in reducing the contingency cost and the incidents of adversary situations resulting in disputes and claims.

The suggested design-construct approach has merit, but is only viable provided the owner understands the part that he has to play in the project and the risks and contingencies that are to his account.

Schedule Control

Schedule is very important since the characteristics of the subsurface conditions dictate that construction be completed in a continuous, smooth fashion. To this end, scheduling of the project should be controlled by computer-analyzed critical path method (CPM). The major elements for this CPM should be listed as well as detailed activities for each of these events. It is important that the working of the CPM get input from the engineer as well as the contractor.

Generation of the critical path for any project is an effective tool in control of the various construction activities from start to completion. The critical path of the CPM program should be as much a part of the planning of the facility as its design and should be worked out in the early stages of the project while subsurface and site conditions are being developed and the project is on the drawing board. Events which may not intuitively seem critical to on-site supervisory personnel at the start of construction can be efficiently evaluated to determine their effect on the total project completion. For example, accelerated or double-shift work may be more advantageous near the beginning of a project if a critical item is behind schedule, as opposed to accelerating construction of a non-critical item near the end of the project. The influence of such decisions, in particular as the subsurface may be distrubed by the excavation process, on the predicted behavior of the facility need to be considered both during the design stages and throughout actual construction in the field.

With an adversary approach which separates engineering from contracting, it is important to note that if a critical path network is independently prepared by the contractor and agreement is eventually reached on a schedule that is similar to one prepared by the engineer, disputes between the contractor and owner (or his representative) would not be anticipated. Continued monitoring of the contractor's performance and updating of the critical path network is needed, however, to assure that critical items are not overlooked and that project progress is not behind schedule.

With the design-construct approach, the CPM would, of course, ideally be generated and modified by the engineer and the contractor in cooperation relative to the predicted behavior of the facility, the construction operations, and the influence that these operations might have on the completed underground project.

Cost Control

Since cost control is essential on any project, it should be computerized to included but not limited to at least the following items:

1. Preparation of a daily foreman's time sheet by construction personnel with proper assignments of time to authorized categories or units of work.
2. Submission of duly processed time sheets to the head accounting office for payroll purposes as well as cost control

3. Preparation of computer master file showing all labor rates, equipment rental rates, unit costs of material, etc., and appropriate allocation to the various categories of construction units.
4. Calculations made daily of the various expenditures for each of the authorized categories or units of work.
5. Preparation of computerized reports to provide construction management staff as well as the owner and/or his project manager with effective daily and/or weekly summaries of all costs and schedules of work with percentage of completion to disclose clearly the construction progress.

By comparing the categorized cost on the cost control side of the program to the categorized quantities on the quantity control side, a direct indication of unit cost for each item of work can be obtained. By comparing this unit cost to the contract unit price contained either in the target estimate, lump-sum bid, or whatever contractual agreement might have been arranged between the engineer-contractor and the owner, a direct indication can be obtained as to whether the estimated or bid price on any particular item of work is being met. If periodic review indicates that an actual unit price is becoming higher than the target or bid price, an evaluation of the construction procedures on that item can be performed to assess the reason for the increase in cost. Modifications can be made to the construction schedule and/or the procedures to minimize future expenditures.

A design-construct approach into which flexibility of contract agreements have been built and which allows for modifications both in the design and the construction procedures, provides one of the better means for cost control and for achieving the target cost for an underground construction project.

Engineering Control

Successful implementation of the design-construct approach depends upon (1) making a detailed prediction of the performance of the facility before construction for a given (measured or presumed) set of conditions, including the soil-structure interaction, the structure itself, the subsurface materials, water conditions, the excavation process, the uncertainties and risks considered in the design within an assigned level of conservatism; and (2) selecting for (1) above, the critical elements that are key in the prediction of the performance of the facility, and the monitoring of these key elements faithfully and accurately during construction.

For (2) to be effective, requires that variations from the predicted movement and behavior of the subsurface materials during construction be detected before distress or difficulties are experienced. The results of the monitoring (measurements and observations) should be interpreted at the site and changes in the construction process should be made and/or terminated in order to maintain subsurface conditions compatible with the predicted soil-structure interaction. Where this is not possible, additional subsurface exploration, testing and gathering of data should be promptly made for the appropriate changes in the design of the facility.

It has been suggested that the design-construct approach using an incentive contract may be a promising way of achieving a breakthrough in underground construction. A model of such a contract has been suggested and it was further indicated that major contingencies may be eliminated through cost and schedule control. A project so conceived might be indeed virtually assured of early completion at or below target cost. The real world is never so simple, however, and "mother nature" has surprises lurking beneath the earth's surface that can be minor distractions at best and unrelenting nightmares at worst.

What makes or breaks a project is not so much what is there, but how well one is prepared to deal with reality when all its ramifications are fully uncovered. If one can predict accurately what the subsurface conditions will be at depth for a particular underground project, surprises could be virtually eliminated. If there are no surprises, schedules should be easy to control. Admittedly this is no easy task, but surely a reasonable goal. There is a need to make better predictions of subsurface conditions and how they will affect support requirements, excavation techniques, etc. These predictions should be evaluated in such a way that we can recognize when we are wrong and that we have an alternate plan ready to put into action if our optimism is not borne out.

To improve underground construction, the design-construct approach (when combined with field measurement and observation) provides a means for maximum economy of construction and assurance of safety of the project since it provides for modification in the design during construction. Certainly, such an approach has little meaning if the designer does not have in mind a plan of action for every unfavorable situation that might be disclosed by the field observations. Equally well, the observations must be reliable, must reveal significant phenomena, and must be so reported as to encourage prompt action. For many present projects, the design-construct approach will not work because of contractual difficulties in implementing required changes, but if the design-construct approach with appropriate field observations is used, it can have significant effects on reducing the final cost of the project and in aiding the construction forces to accomplish their work.

If the worst is planned for, a conservative design results. An alternate would be to assume reasonable conditions and be prepared for the worst; thus, contingency funds would be used only if necessary. The key element in the design-construct approach, for its efficient and effective use of evaluating subsurface conditions as exposed during construction, requires a flexible contract that permits required changes to be made promptly. It must be emphasized that these changes require that simple, practical and reliable measurements and observations be made during construction and that the data be constantly evaluated and interpreted.

SUMMARY AND CONCLUSIONS

An incentive contract within a design-construct approach has to be one that permits innovation and exercise of creativeness during construction. Innovation should be given every opportunity to flourish and the owner should be afforded the opportunity to gain savings once the risks, if any, with the innovative process have been explained relative to the predicted performance of the facility. Innovation cannot succeed without dealing with changes that occur during construction. The contract requires flexibility. No where is it less desirable to separate engineering and construction than in underground work, for the design is not really finished until the facility is built. Admittedly the controls required (quality, cost and schedule) to implement effectively such a contract will themselves require a great deal of thought and planning.

The element of competition can be interjected into a design-construct approach with an incentive contract, but this competition should come during the prequalifying stages in the selection of the design-construct team. Indeed, competition properly controlled will afford owners the opportunity to obtain a better facility for their money. Competition forces those in the business of designing and constructing underground facilities to examine continually their methods and procedures and to reward innovation, effectiveness and efficiency. A major challenge, however, is to provide the necessary control with creative, flexible and fair contract documents that not only specify a final product, but permit the product to be safely achieved via the most economical route and on schedule.

A fair contract for underground engineering and construction is one where the owner, the engineer and the contractor have considered the uncertainties, and the risks with the appropriate contingencies have been properly defined and assigned to each party involved with the project. Because of the interplay and interweaving of the uncertainties associated with subsurface conditions, the combining of the uncertainties, risks and contingencies involved in engineering and construction and assigning these to a design-construct team provides one of the fairer approaches for the building of underground facilities.

The construction industry is generally plagued by inefficiency and lack of flexibility that has resulted in some spectacular cost and time overruns. One of the inherent problems in the industry is having to deal with subsurface conditions that cannot be completely known prior to construction. The projected dollar volume of underground construction in the next decade makes it vital to seek immediate ways to lower the cost of construction. Clearly, a breakthrough is needed. Of the many possibilities of lowering the cost of underground construction, a contract that permits engineered-construction to be achieved is a promising step in the right direction. A design-construct negotiated incentive contract that rewards innovation and construction savings has been suggested. It has been strongly added for the design-construct approach to have a reasonable chance of success, controls are needed for contingencies, cost and schedule.

The time is here now to apply the proposed design-construct approach or engineer-construct method to the metro construction in our highly urbanized areas. To reduce costs of underground construction requires, with the proposed design-construct method (and its apparent promise), a change in the procurement policy of both government and private industry and a new outlook by the engineering profession and the construction industry.

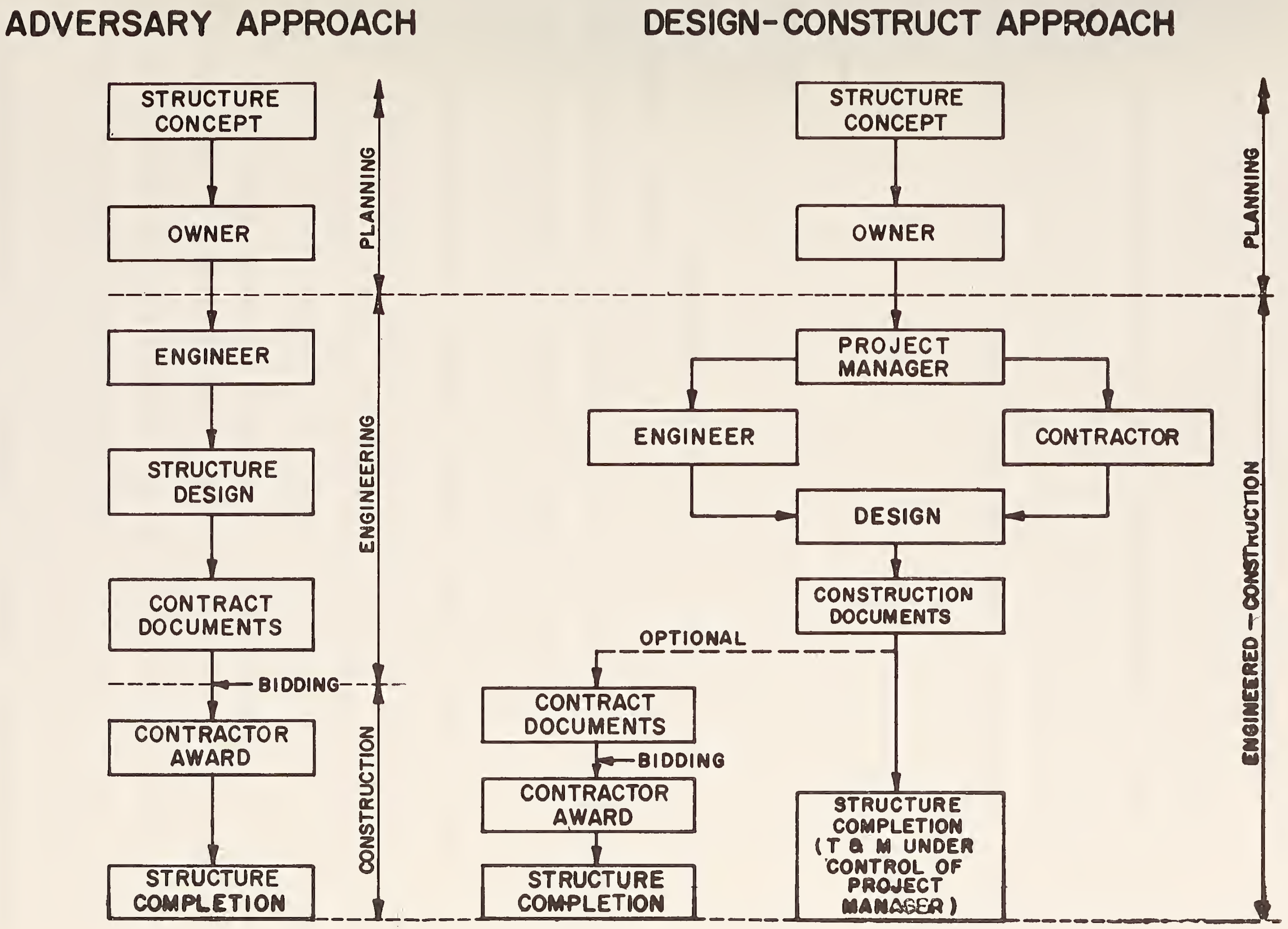


FIGURE I – TWO DIFFERENT APPROACHES FOR ENGINEERING AND CONSTRUCTION OF UNDERGROUND FACILITIES

IV. THE PREFABRICATED PANEL SLURRY WALL AND ITS APPLICATIONS

Since its first full scale application on a construction project in 1970, over one million square feet of prefabricated diaphragm walls (PANOSOL walls) have been installed using the slurry trench method. This paper, which is adapted from a lecture in French by G. Y. Fenoux who is the head of Soletanche's design department, describes the method and its applications in the field of diaphragm wall construction.

Introduction

Cast-in-place concrete walls constructed using the slurry trench method have been used successfully for more than 12 years. The so called "slurry wall" technique is now more or less routine in civil engineering practice and installations are now relatively commonplace.

Construction of a classical slurry wall involves three operations: trenching, placing rebar cages and tremie concreting. Over the years, the technology and equipment for trenching under bentonite slurry have been perfected to the point where trenching operations proceed smoothly and without incident in most all types of ground. Placing the reinforcing bar cages in the trench has never been a major problem. However, the concreting operation has always presented difficulties. Once the concreting of a panel begins, the operation and the integrety of the completed panel is at the mercy of the smallest incident: a bad batch of concrete, breakdown at the batching plant, breakdown of delivery trucks, cranes, etc. On urban sites, the problems are increased by traffic and working hours limited because of noise. In spite of the progress made during the past twelve years, concreting a classical slurry wall remains a delicate operation and one that is difficult to control precisely. In addition to the concreting operations, there can be difficulties with the joints between panels; none of the many techniques for making joints are entirely satisfactory.

In 1970 Soletanche began experimenting with precast concrete panels thereby eliminating the problems associated with concreting in-situ. The use of prefabricated panels enables the primary criteria -- quality, safety, and control -- to be satisfied, while at the same time achieving other design and construction advantages associated with precasting.

The concept of excavating a trench and inserting a prefabricated wall element is simple enough. The major difficulty involves maintaining a structural and watertight seal between panels and the natural ground. During the first trials, the seal was attempted by grouting with lean concrete. It was soon apparent that the grouting method would not work effectively. The solution ultimately adopted is to immerse the panels in a trench filled with a slurry that sets to form the required seals. A mix was designed such that it would also fulfil the function of the bentonite slurry during trenching, i.e. support the sides of the trench.

The special slurry mix is composed of water, bentonite, cement and additives, making what is called a self-hardening "coulis". Figure 1 shows the development of mechanical characteristics of a typical coulis as a function of time after mixing. The setting time must be controlled within narrow limits so that the slurry does not set until the excavation is finished and the precast panel positioned in the trench, yet the slurry must gain strength rapidly thereafter. The final compressive strength of the coulis is determined primarily by the cement content of the mix and typically is on the order of 100 to 200 psi.

Two general panel configuration have been developed. The first type, shown in Figure 2 consists of identical panels connected with tongue and groove joints. The second type, shown on Figure 3 is composed of beams and slabs; in this configuration, the slabs bear on beams that are extended below the bottom of the excavation in order to mobilize passive resistance.

PANOSOL construction is quite simple, and involves fewer construction operations, than classic cast-in-place diaphragm walls. The trench is made with the usual equipment and kept full with the coulis slurry during the trenching process. The prefabricated elements are then lowered into the slurry-filled trench using a simple device to assure proper positioning of the suspended panels until the slurry has set. The trenching can be carried out in a continuous line, eliminating the need for primary and secondary panels and cold joints in the hardened coulis.

The general process lends itself to several applications, other than diaphragm walls. Figure 4 shows three: prefabricated joint beams in a conventional slurry wall, prefabricated piles and lagging (called a "Berlin-Wall"), and a "Parisian Wall" where the continuity of horizontal steel is maintained in a pile and lagging type of wall.

Initial Applications of PANOSOL as Earth Retaining Walls

The first project where PANOSOL was used in 1970 was a test site at Blvd. de L'Hôpital in Paris. The project involved a permanent retaining wall for a building with two basements. With the client's agreement a few elements of prefabricated wall were substituted for a section of conventional diaphragm wall. The prefabricated panels measured 8 m (25 ft.) long. This first application was successful as seen in Figure 5.

The first complete job was undertaken in the fall of 1970 at Issy-les-Moulineaux, in France. The project involved an excavation of three basement levels for the offices of Electricite de France. The system of beam and slab construction was used, with the longest elements 15 m (50 ft.) in length. The beam and panel configuration is aesthetically pleasing and is easily adapted to various lateral loading conditions as shown in Figures 6 and 7. The beams can be moved closer together or further apart by adjusting panel width, thereby altering the required reaction due to tie-backs, bracing or toe resistance.

PANOSOL Applications Involving Composite Walls

Generally speaking, a diaphragm wall has three potential functions: retaining wall, seepage cut-off and bearing element for both temporary and permanent conditions. In the cases cited previously, the PANOSOL walls served primarily as retaining walls. In other cases, several functions may be combined economically by using a composite wall.

The principle of a composite wall is illustrated by the case shown in Figure 8 and 9. The upper part of the wall serves to retain an excavation of two basements, while the lower part forms a watertight seal in conjunction with the naturally impermeable sub-strata thereby, establishing a watertight enclosure. It should be noted that the water table could be permanently lowered in the interior of the enclosure by installing a small number of relief wells connected to a sump to collect and remove the small amount of underseepage. The basement floor mat was therefore thinner because uplift was eliminated and did not need to be watertight. If a conventional slurry wall had been used, concrete would have been poured for the full depth of the trench. The PANOSOL process offered a different solution because one constant and essential property of self-hardening coulis is its impermeability. Therefore, it sufficed to trench using coulis down to the substrata and to place the prefabricated panels only in the upper part where structural strength was required for the retaining wall function. The coulis at the bottom, with its low structural strength, served only as a watertight seal.

It is important to note that the economics of this solution result not only in savings in materials, but in lower labor costs. What would have been a lengthy concreting operation was considerably shortened. A photograph of the wall, taken when the excavation was nearly complete is shown in Figure 10.

The above job, an apartment complex called "Le France", was completed in the spring of 1971. Immediately after, a job called "Tour Horison", part of the same complex, began on the Quai National in Paris. The technical design problems were much the same and a similar solution was adopted. The project has three basements as is shown in Figures 11 and 12.

Figure 13 shows the location of the two jobs discussed above: "Le France" (now called "Tour Blanche") and "Tour Horizon" and a third complex the "Centre Cegos". For this later development, the prefabricated panels provided the retaining function on top (for a three story basement) and the seepage cut-off function below. However, in addition to these two functions, the slurry wall was also used for a third function, that of a bearing wall. For this reason, trenching continued through the clay down to the underlying beds of limestone and marl (Figure 14). In the lower portion, bearing stresses are supported by the coulis; the average vertical stress, calculated over the width of the trench, is about 100 psi or about 14 tons/lineal ft of wall including the weight of the panel.

This PANOSOL was the first to be loaded axially, so the movements were closely monitored. Settlements measured on benchmarks situated in the bottom basement have been between 1 and 3.5 mm (approximately 0.1 in.). The settlements may be so small for two reasons: the coulis has a higher modulus than expected because it is mixed with sediments from the trenching operation; a large part of the resistance is taken in side friction over the height of the panel between the bottom of the excavation and the bottom of the concrete elements.

A second example of a PANOSOL wall used as a bearing element is the retaining wall for two basement levels around a complex situated on L'Ile de la Jatte in Paris (Figure 15). To obtain adequate bearing capacity, the wall was socketed into the marl layer. The solution selected involved lowering the water table permanently by collectors and sumps under the basement slab. To reduce the residual underseepage from the limestone, the limestone immediately below the wall was also grouted. Figure 16 shows the sequence of construction: excavation leaving a peripheral berm, construction of the basements in the central portion, bracing of the retaining wall against the completed portion of the building, removal of the berm, and the completion of construction of the basement (Figure 17).

In the two preceding examples where the PANOSOL wall assumed the three functions -- retaining wall, seepage cut-off and bearing element - the compressive stresses in the coulis were relatively small and in the range of 100 psi, so the coulis was able to support the load. For higher bearing loads a different solution is required. For example, at the complex called Courcelles-St. Honoré in Paris, PANOSOL was used to support an excavation for four basements as shown in Figure 18. The PANOSOL sustains a vertical load imposed by the bearing walls of the eight to ten story superstructure. The bed of Beauchamp sands in which the wall was to be founded does not have adequate capacity to support the loads; the obvious bearing layer was the marl some 5 m (15 ft) further below. One possible solution was to trench down to the bearing layer all along the perimiter of the wall. Another solution was to employ a common technique used with conventional slurry walls, nicknamed "the trouser legs" and illustrated in Figure 19, to save as much trenching as possible. This technique involves extending a number of panels for example, every third panel, to a bearing layer. The second solution was chosen; the bearing elements (the "legs") were filled with tremie concrete. It should be noted that the use of "trouser legs" is possible when there is no water seal function required in the lower part of the trench. Figure 20 shows a portion of the exposed wall.

PANOSOL Applications in Public Works

The PANOSOL technique has been applied to numerous Public Works projects. The first application was for the thermal powerplant at Porcheville built by Electricté de France, where since 1971, PANOSOL walls have been used for the excavation for the generator foundations and for the cooling water canals (Figures 21 and 22). Note in Figure 22 that the starter bars at the top of the panels were bent outwards and fixed to the floor slab reinforcing to eliminate the need for temporary bracing at the top of the wall.

A second and more characteristic example is the depressed and covered portion of an extension to Highway A13 between the St. Cloud tunnel and the Auteuil interchange on the ring-road around Paris (Figure 23). The roadway was constructed using cut-and-cover techniques over a length of 800 m (2500 ft) in front of Ambroise Paré Hospital. Figure 24 shows a cross section of the project. The construction sequence was the following: construction of the lateral walls and central supports, pouring the slab that formed the cover, restoration of the surface and excavation for the roadway working under the roof slab. The advantages of this method include rapid reestablishment of surface facilities across the construction area and protection from the environment for excavating and finish work. The side walls combine three functions: retaining, water seal and bearing element; the central supports are used for bearing only. The exterior side walls consisted of identical panels forming a continuous wall, while the central panels were installed with three meter (ten feet) gaps between adjacent panels. The vertical load on the side walls was 25 kg/cm^2 (700 psi) while the vertical load on the central panels was approximately twice this amount. As shown in Figure 25, the central panels were extended to greater depth in the chalk because of the higher bearing stress. Four possible solutions were investigated for transferring the load carried by the panel to the bearing material.

Solution 1 - Use the general coulis technique with additional cement (the simplest solution.) This solution requires enough cement in the coulis to obtain a 28 day strength of 150 to 200 kg/cm^2 (2100 - 2800 psi). Such a coulis is quite dense and would become loaded with sediments from the trenching operations. Trenching and placement of the panels would be difficult and precision would be hard to control.

Solution 2 - Use a substitution technique. Trench with bentonite slurry and then change this slurry for coulis having the required strength in the lower portion of the panel. This solution requires desanding the bentonite slurry used during trenching; otherwise the two liquids of similar density in contact would mix. Also, continuous trenching would not be possible, because a panel would have to be allowed to set before trenching the adjacent panel or else the different fluids would mix, contaminating the coulis.

Solution 3 - Use a combination of the two preceding solutions, i.e. excavation with a coulis of normal strength (to seal the panels together and with the soil) and filling of the lower part of the panel with a high strength coulis. Continuous trenching would be possible if certain precautions were observed.

Solution 4 - Go one step further with the preceding solution and exchange the lower portion of the coulis with tremie concrete.

Solutions 1 and 2 were eliminated; Solution 3 was acceptable for the side walls but less satisfactory for the more heavily loaded central suppo[illegible]lution 4 was chosen.

It was essential to assure a good contact between the tremie concrete and the prefabricated panels. As a precaution at the beginning of the job, three vertical pipes were cast into the panels as shown in Figure 26: a central tube large enough to lower a concrete vibrator into the tremie concrete and two side tubes to allow future grouting of the contact. The use of these pipes was never required. In fact, the job was organized so as to insure rapid and precise placement of the panels. The speed of execution was important because the panel had to be placed while the tremie concrete was still fresh. Several core borings were made to verify the quality of the contact between tremie concrete and the precast panels. The results were very satisfactory as shown in Figure 27 and the pipes were eliminated for the rest of the job. Figures 28, 29 and 30 show several views of the job.

On the basis of the successful results achieved on A13, a very similar project was undertaken in Zurich, Switzerland. As shown in Figures 31 and 32, PANOSOL walls were used on over 1 km subway line and station. The panels serve as the permanent finish walls, even in the central station. This job presently holds the record for the number of panels, over 1,100.

V. Watertightness of Slurry Walls

An essential quality of slurry walls used for permenent construction is watertightness. A conventional slurry wall has the reputation of being watertight. However, tremie concrete in spite of the high cement content (cement/water ration usually about 2), cannot be considered completely watertight with the usual two feet thickness. Seepage occurs both through the concrete and at the joints. When the exterior water head exceeds about 5 m (15 ft) it is not entirely possible to avoid damp spots on the wall and tiny leaks. These become evident especially when the basement ventilation is poor. Tremie concrete is too permeable to prevent damp spots on the walls without a liner. The liner can be, as a resin paint or mortar applied to the exposed surface of the wall.

PANOSOL walls improve watertightness significantly. The concrete used for prefabrication is less permeable than tremie concrete. The layer of coulis on the soil side of the panel provides additional waterproofing. Moreover, it is possible to place a low permeability layer on the soil side of the wall, for example, bituminous paper. Sandwiched between the coulis and the concrete, the waterproofing works under ideal conditions. To maintain continuity of the layer it is necessary to treat the joints between panels because, exposed to the air, coulis has a tendency to dehydrate and crack. The joints are several centimeters thick and treating them is not simple because a good joint must have the mechanical strength to resist water pressures, and must be able to deform plastically to follow any differential movements between adjacent panels.

A number of products have been tried to obtain satisfactory seal of the joints between panels. The subway authority for the city of Lyons in France made a test section of prefabricated panel wall for the future Lyons subway. This job provided the opportunity to test some sixty different joint treatments under particularly severe water conditions. Several types of joint treatments proved satisfactory. Without entering into specific technical details, the best type of joint treatments are composite, made up of a resistant cord for strength, a plastic watertight portion and an architectural facing.

Outlook

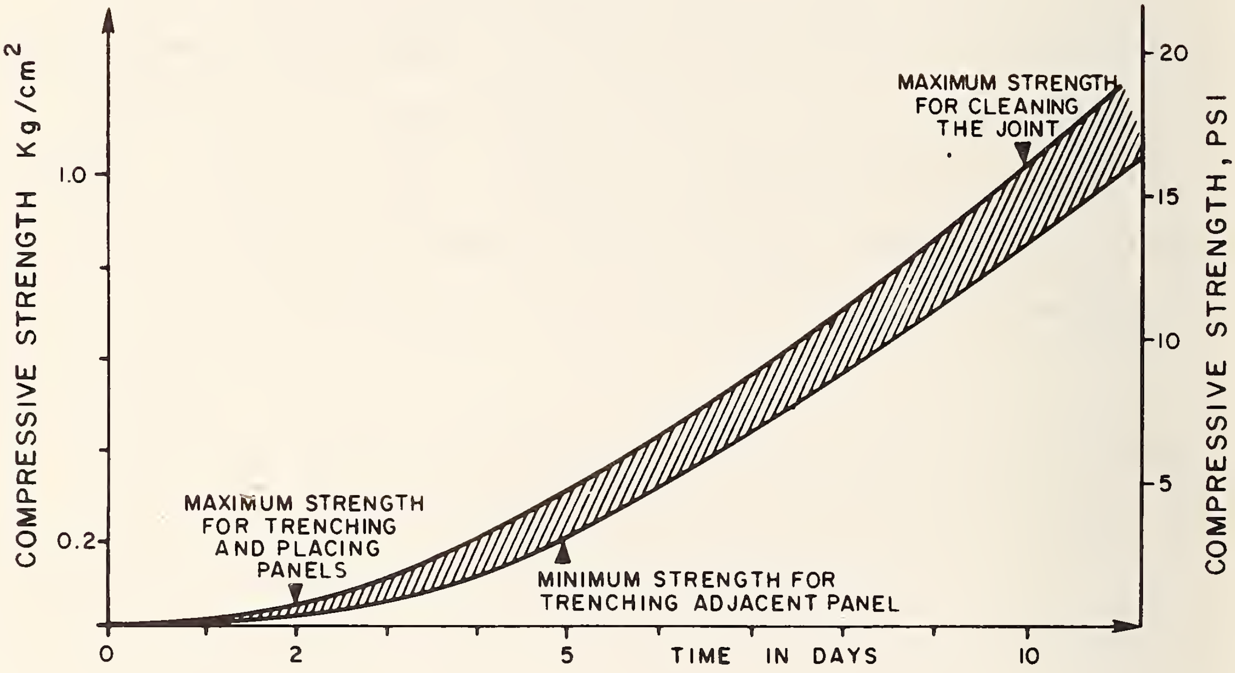
At the present time the practical limit of the length of PANOSOL walls is determined by the weight of the prefabricated units. A maximum panel weight of about forty tons permits excavation for four to five basements; this upper limit will surely be improved.

Up to the present time, panels have been almost exclusively reinforced concrete. Research into other materials such as liminated wood and metal have not, for the moment, produced definitive results. In the realm of concrete, progress is still possible employing the use of light weight aggregates and hollow panels. In the realm of reinforcing, prestressing remains to be tried. (Prestressed bonded bars were already tried on one small job site).

So far the advantages inherent in precasting have not been mentioned: quality control of the concrete and the covering of the reinforcing cages, appearance, precision of starter bars and reservations, and increase in space within the resulting excavation due to thinner walls. In the urban environment, construction with prefabricated panels offers important advantages: no bentonite mud to dispose, no concrete trucks to rely on, and no joint tubes to extract late at night. The disadvantages are that a job must be large enough to offset certain fixed costs associated with precasting -- the special forms in particular.

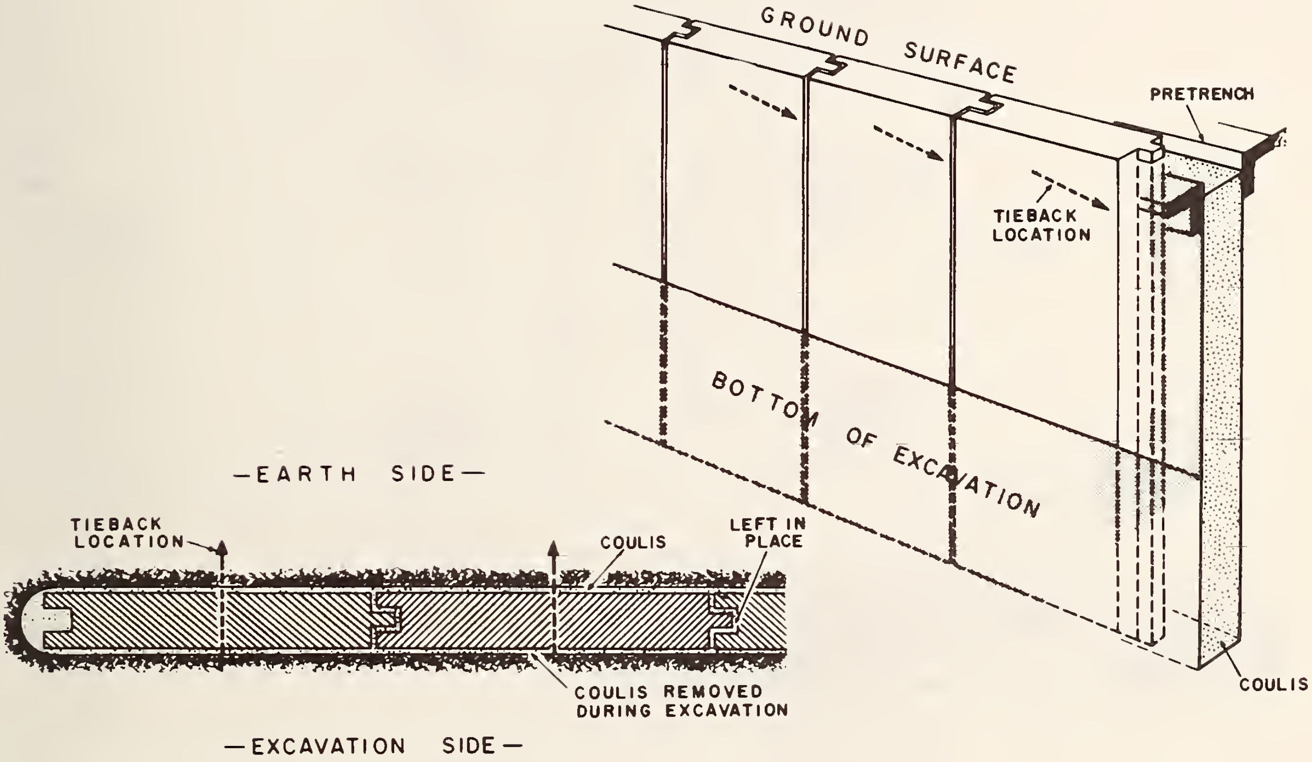
At present, the cost of conventional cast-in-place concrete wall is about 10 percent less than a prefabricated wall under the same conditions. It should be noted that this comparison is not necessarily valid because on one hand, there is a relatively finished product, the prefabricated wall, and on the other hand, a product not usable without additional work -- chipping, cleaning and waterproofing. For certain applications, for example a garage wall in a basement, a rough appearance is acceptable and the cast-in-place wall will suffice. In other cases, if the additional work is accounted for, the difference in cost diminishes. Nevertheless, as experience has been gained with PANOSOL, the cost has been steadily decreasing to the point where PANOSOL is competitive with conventional slurry walls.

A composite wall, retaining at the top and water seal at the bottom, increases in economy as the depth of the water seal increases, without even counting the costs of additional work required on conventional slurry walls. The gain is therefore two-fold: quality and price. Composite PANOSOL walls are usually significantly less expensive than the available alternatives.



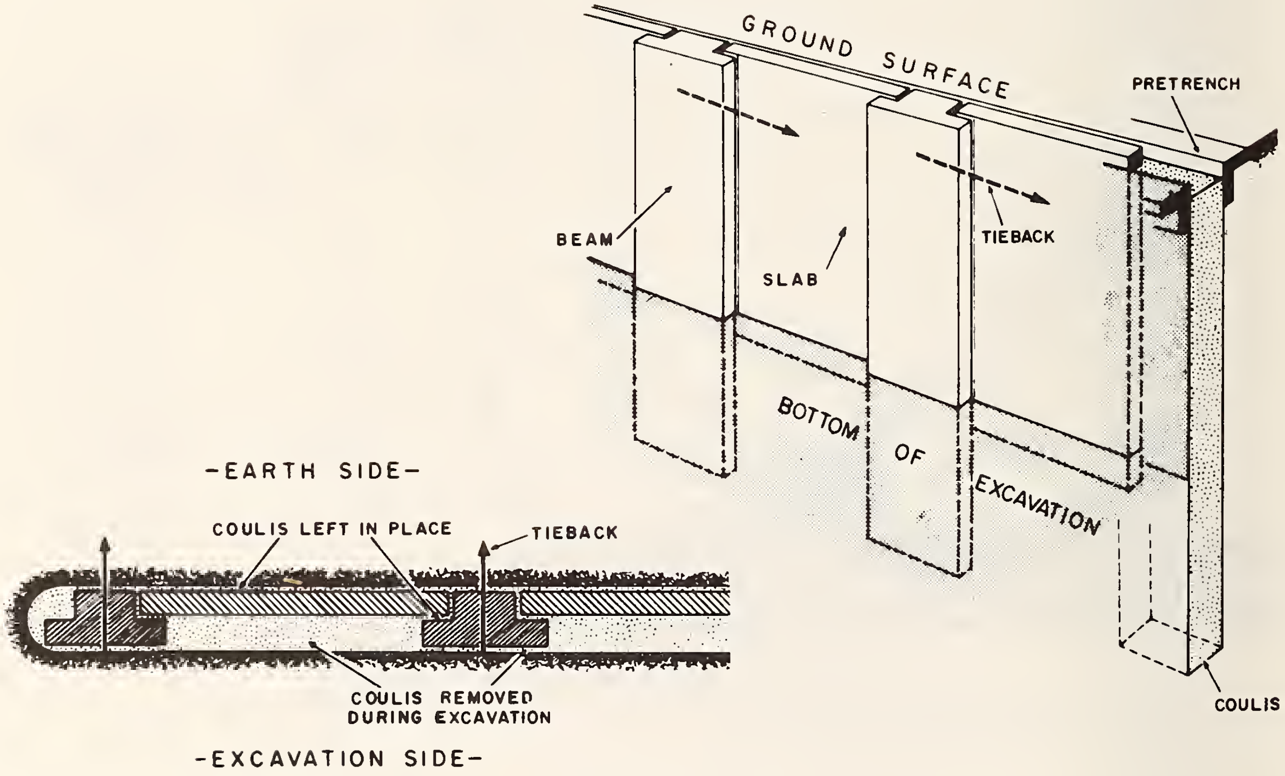
COMPRESSIVE STRENGTH VS. TIME OF A TYPICAL COULIS

FIGURE I



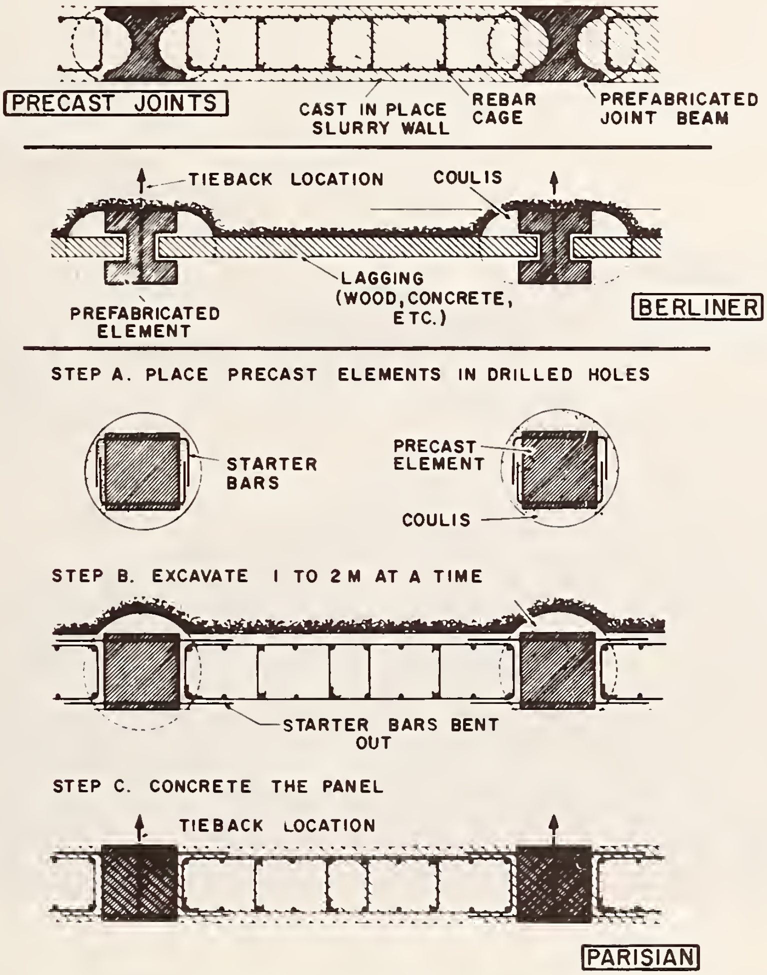
SCHEMATIC VIEW OF PANOSOL WALL CONSTRUCTED USING TONGUE-AND-GROOVE PANELS

FIGURE 2



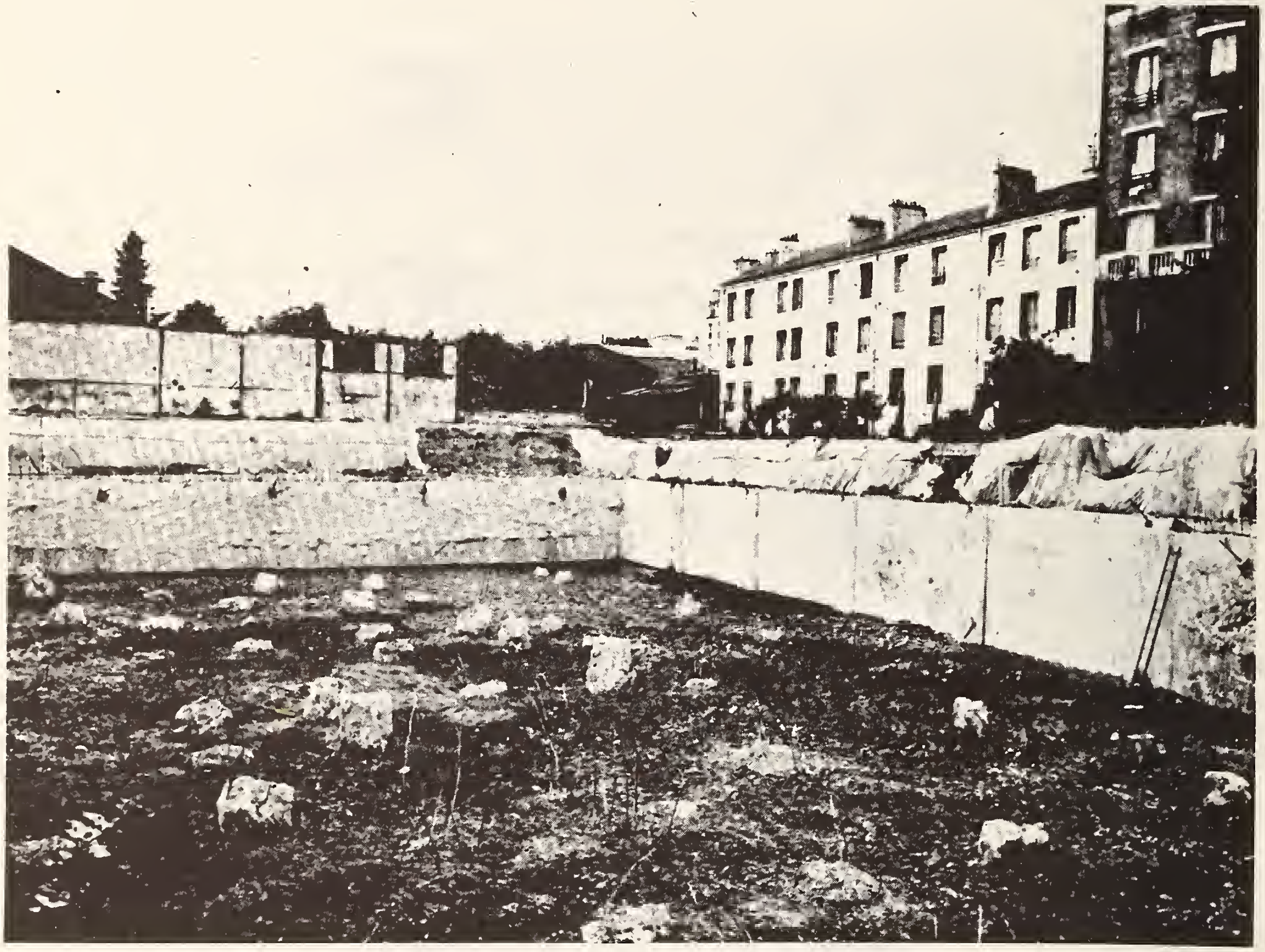
SCHEMATIC VIEW OF PANOSOL WALL CONSISTING OF BEAMS AND SLABS

FIGURE 3



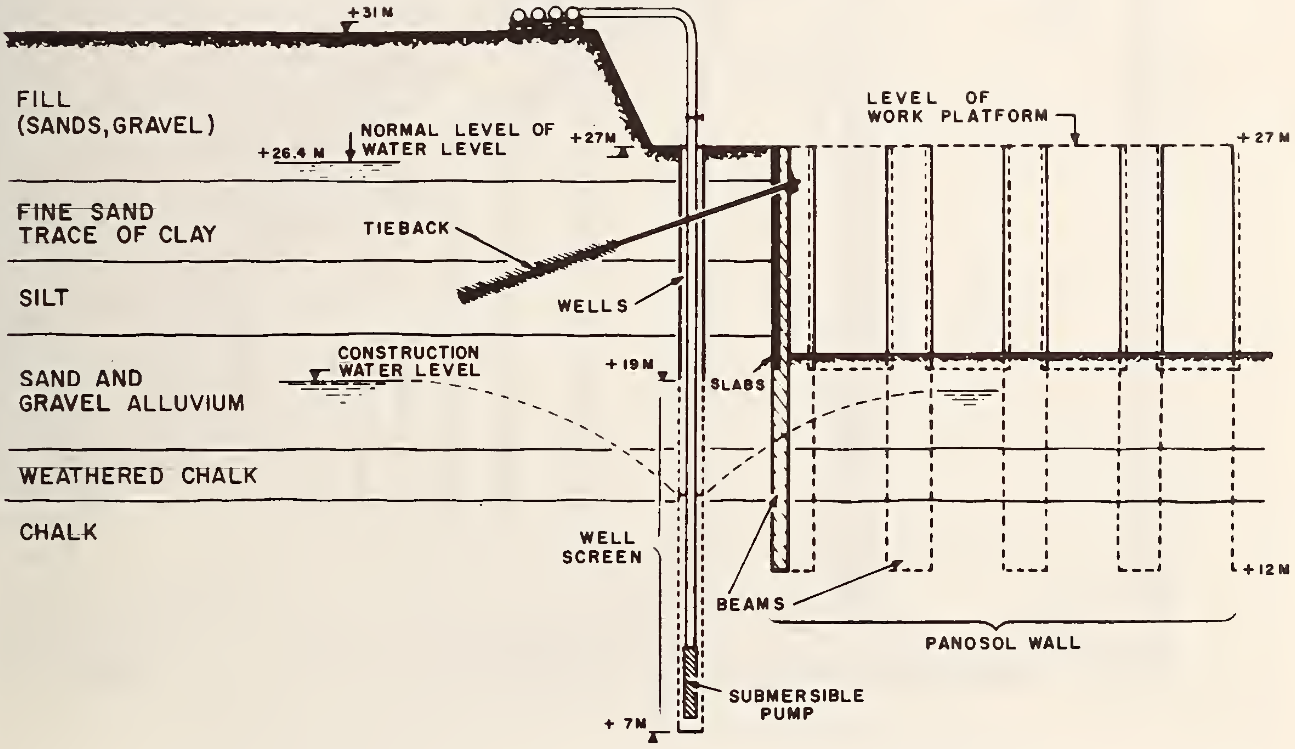
MISCELLANEOUS APPLICATIONS OF THE PANOSOL METHOD.

FIGURE 4



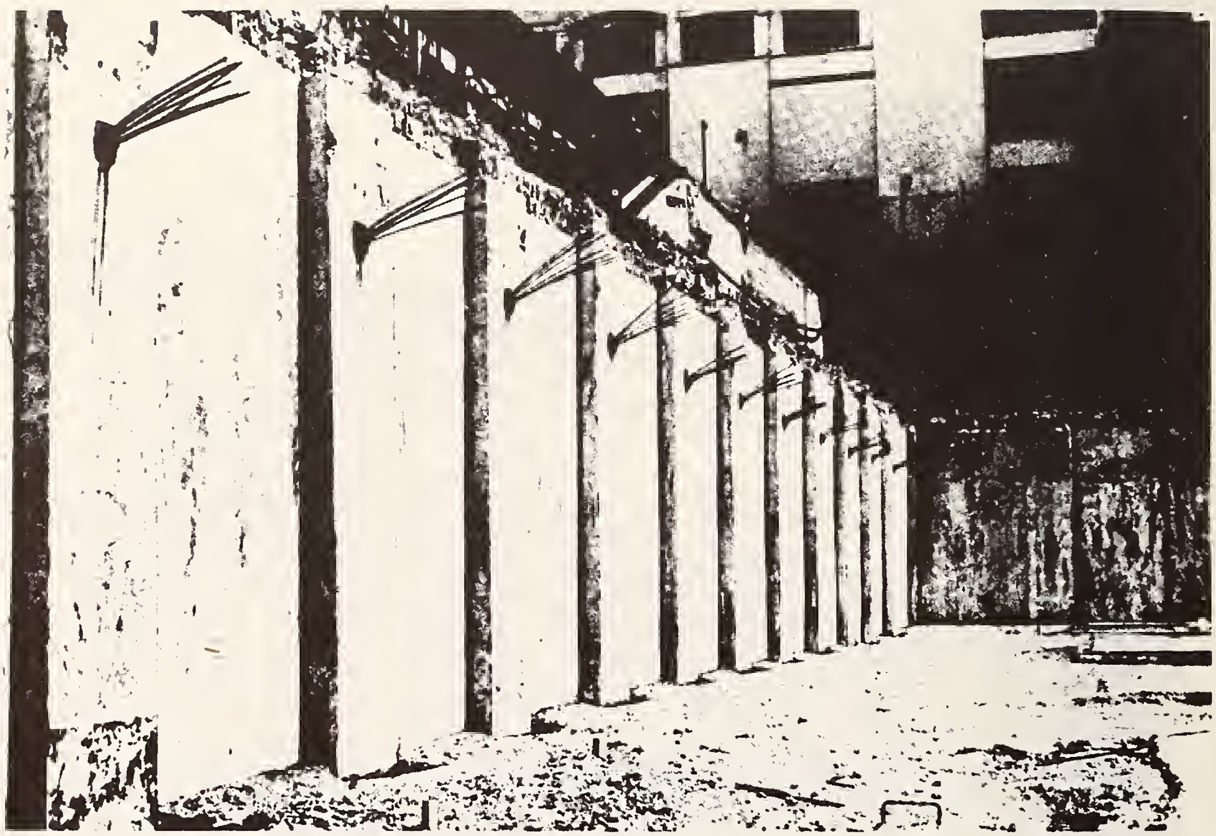
VIEW OF SITE AT BOULEVARD de HOSPITAL IN PARIS

FIGURE 5



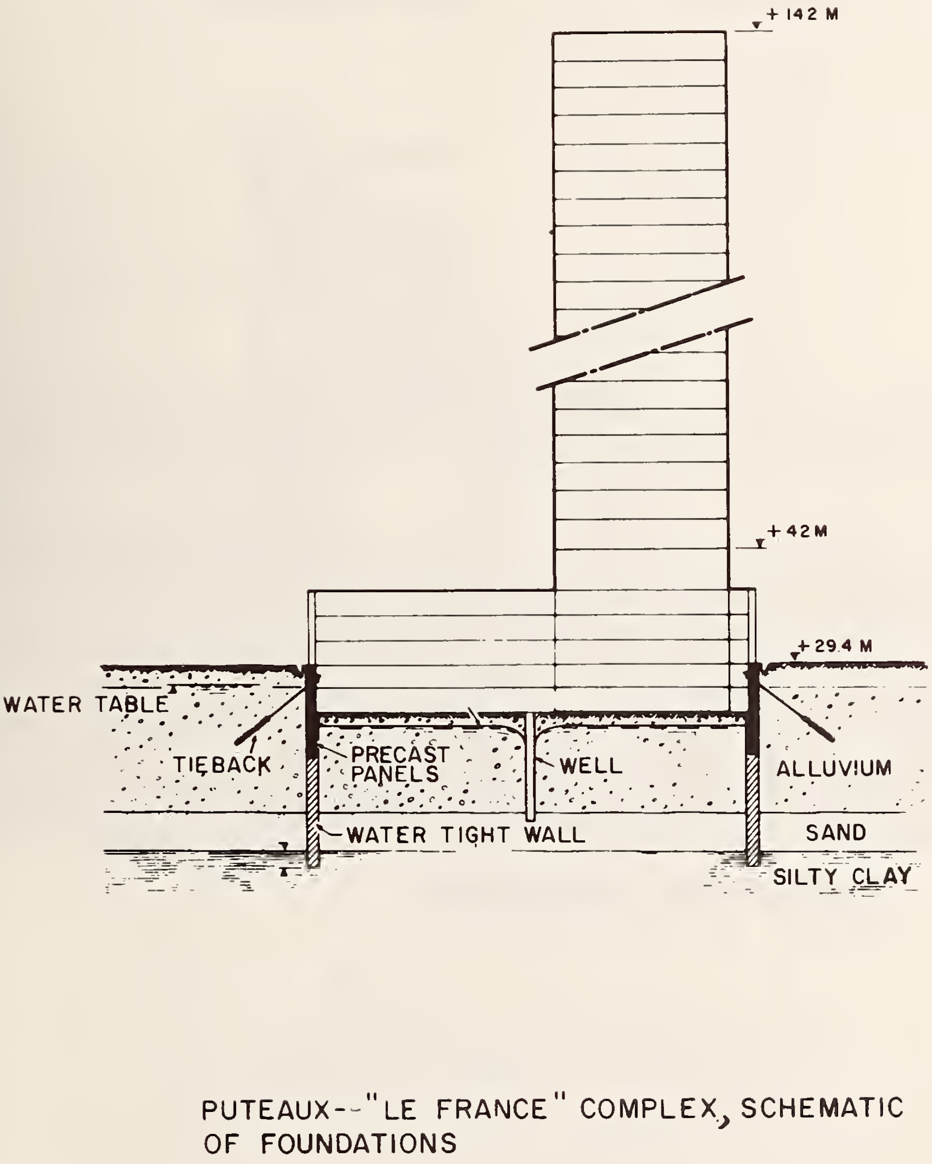
TYPICAL SECTION OF THE PANOSOL PROJECT AT ISSY-LES-MOULINEAUX

FIGURE 6



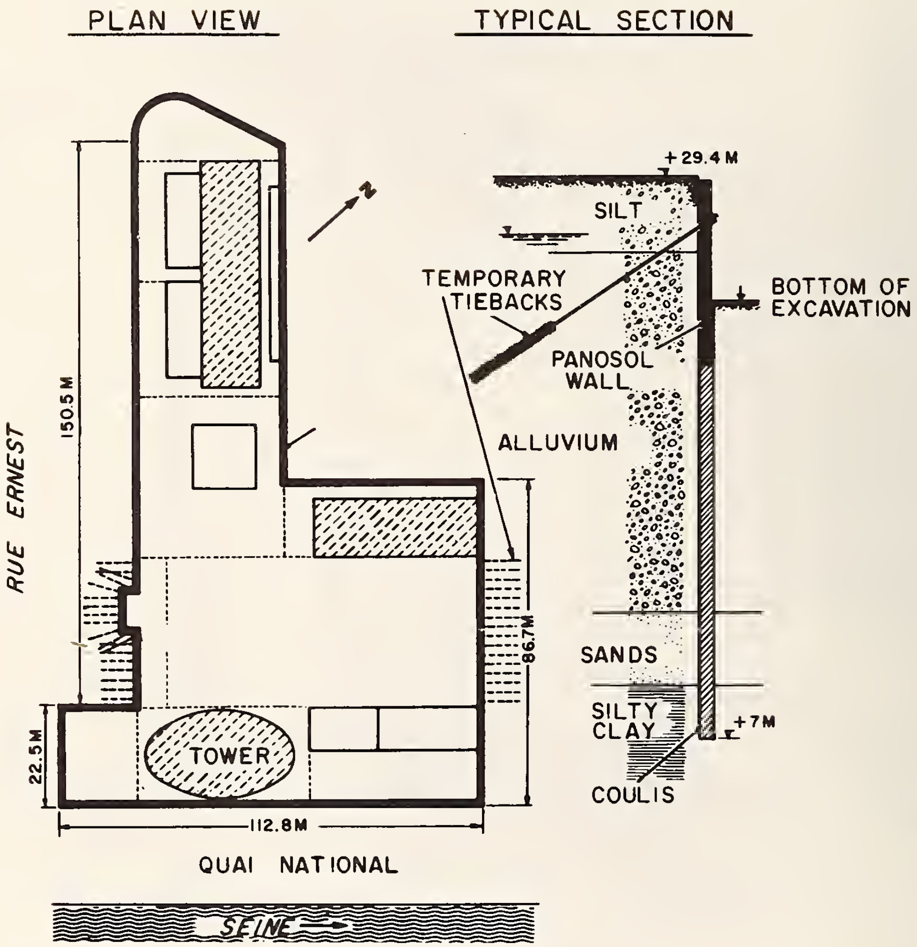
VIEW OF PANOSOL WALL
AT ISSY-LES-MOULINEAUX

FIGURE 7



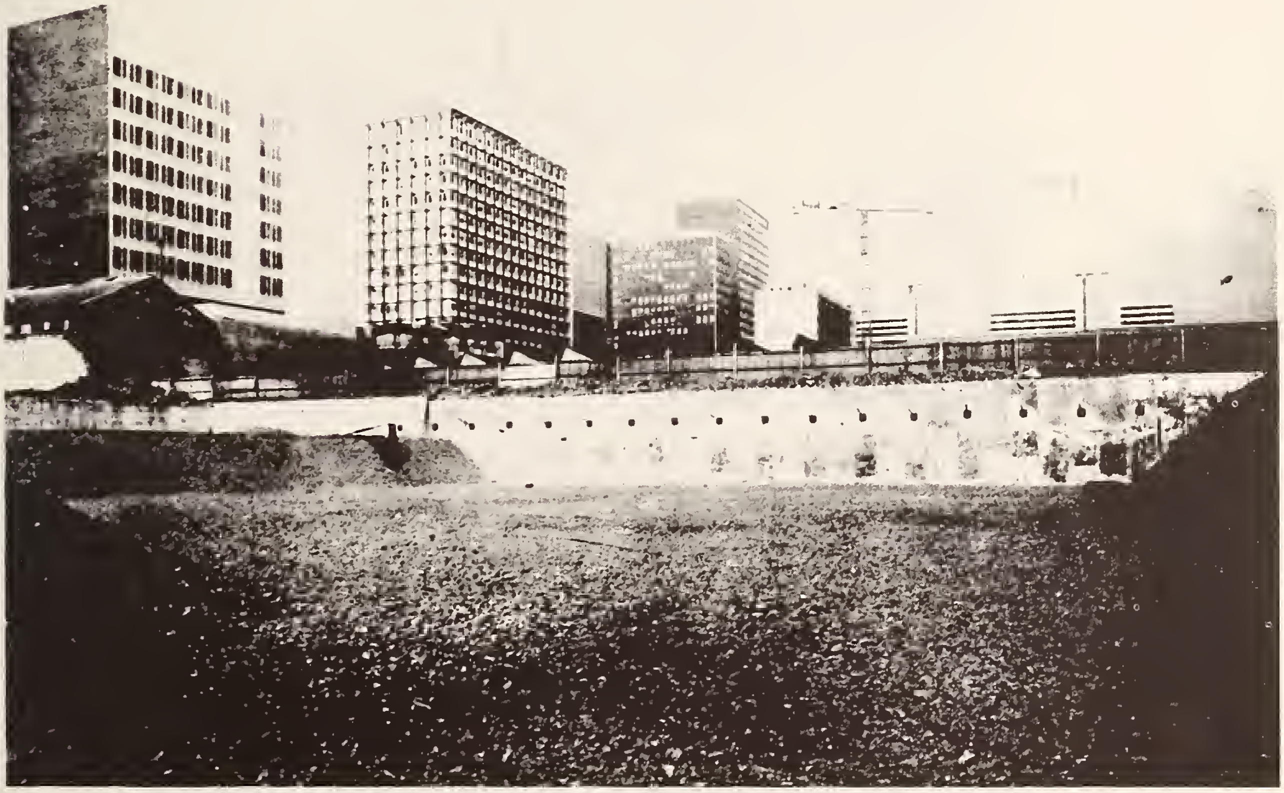
PUTEAUX--"LE FRANCE" COMPLEX, SCHEMATIC OF FOUNDATIONS

FIGURE 8



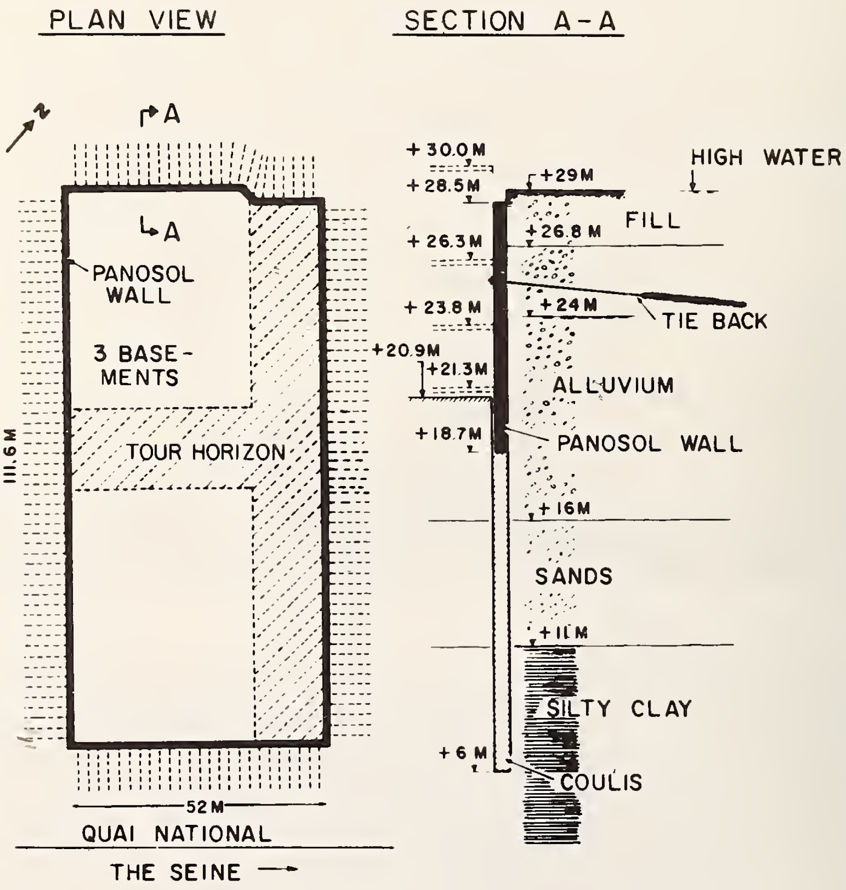
PUTEAUX-"LA FRANCE" COMPLEX.

FIGURE 9



VIEW OF PANOSOL WALL
AT le FRANCE SITE IN PUTEAUX

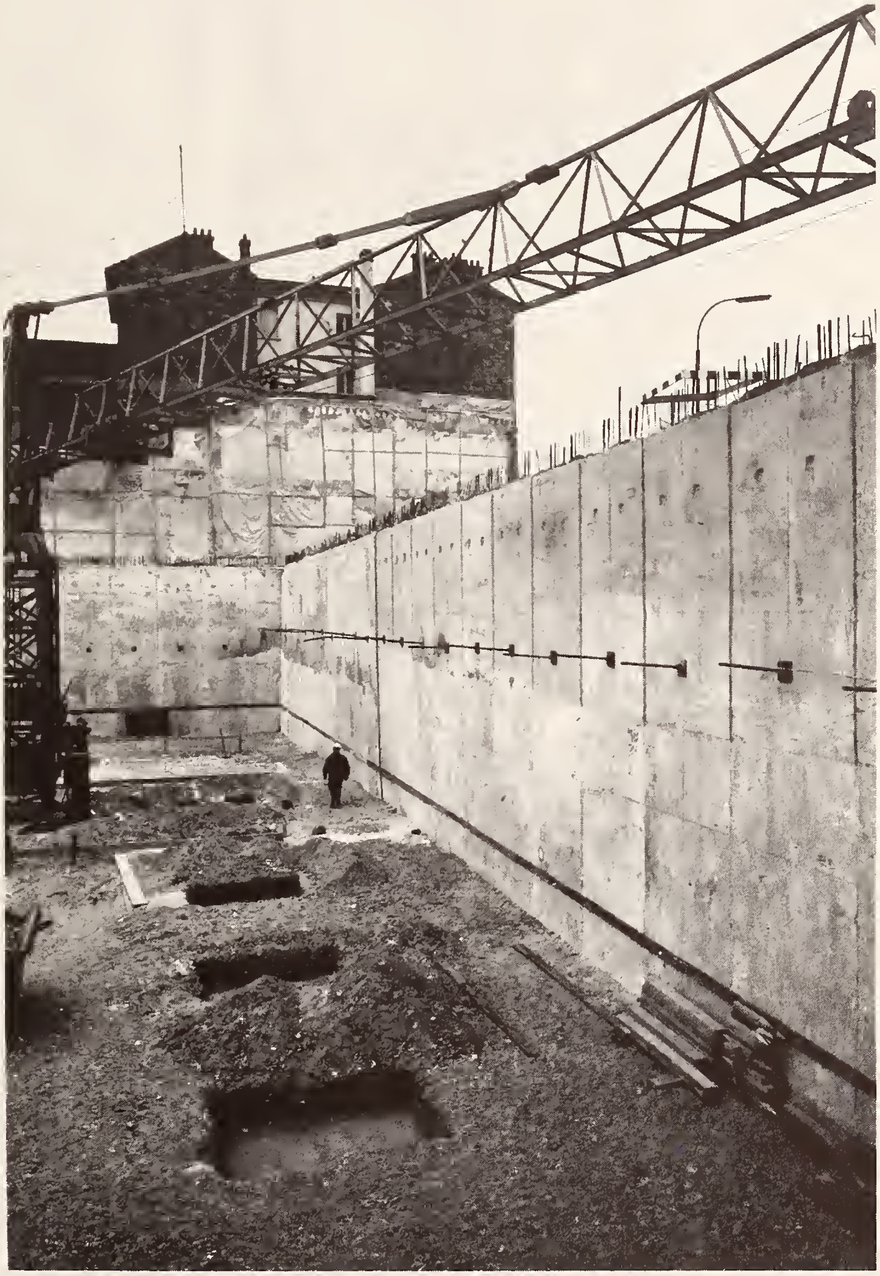
FIGURE 10


PLAN VIEW

SECTION A-A

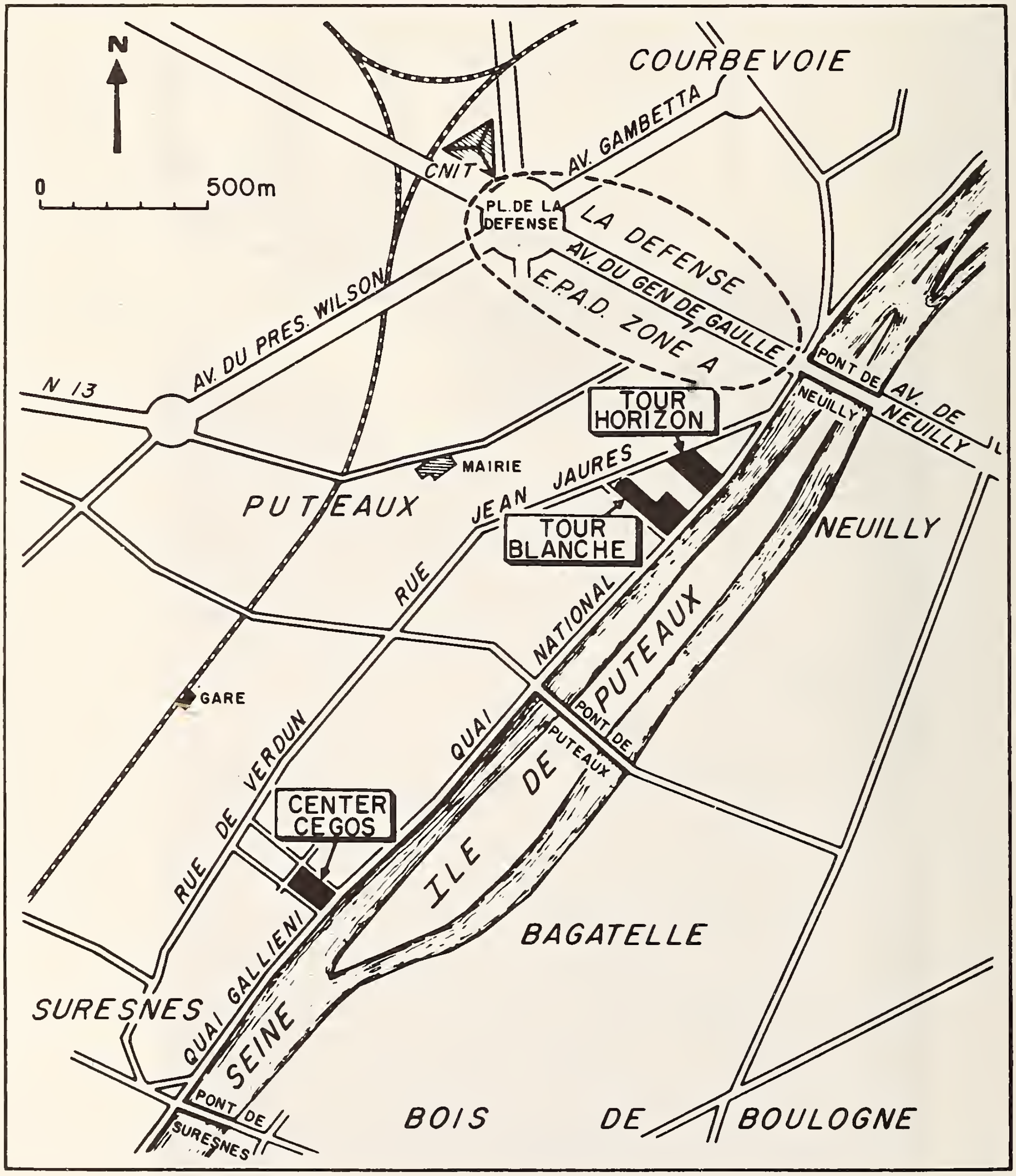
PUTEAUX _ "TOUR HORIZON" COMPLEX

FIGURE 11



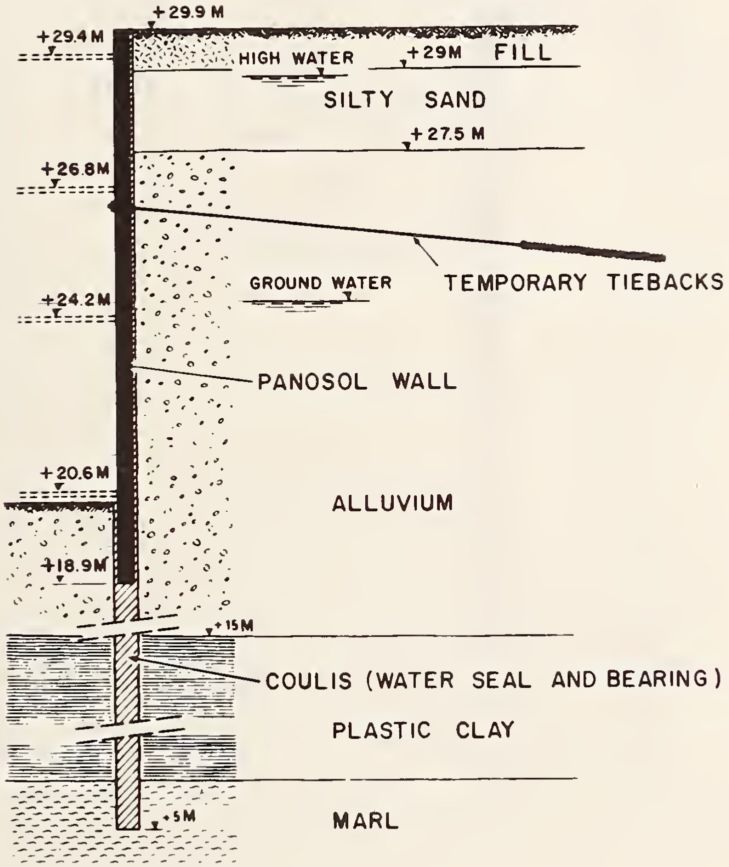
VIEW OF PANOSOL WALL AT
TOUR HORIZON SITE AT PUTEAUX

FIGURE 12



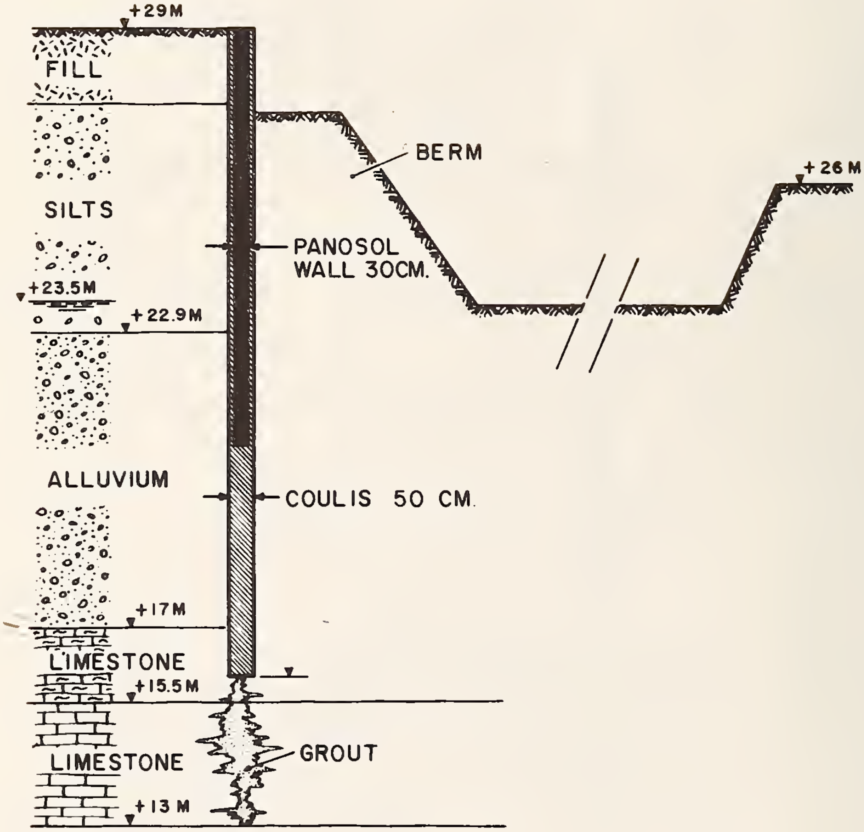
MAP SHOWING THE LOCATIONS OF THE THREE PANOSOL PROJECTS: "LE FRANCE" (FORMERLY "TOUR BLANCHE"), "HORIZON", AND "CEGOS".

FIGURE 13



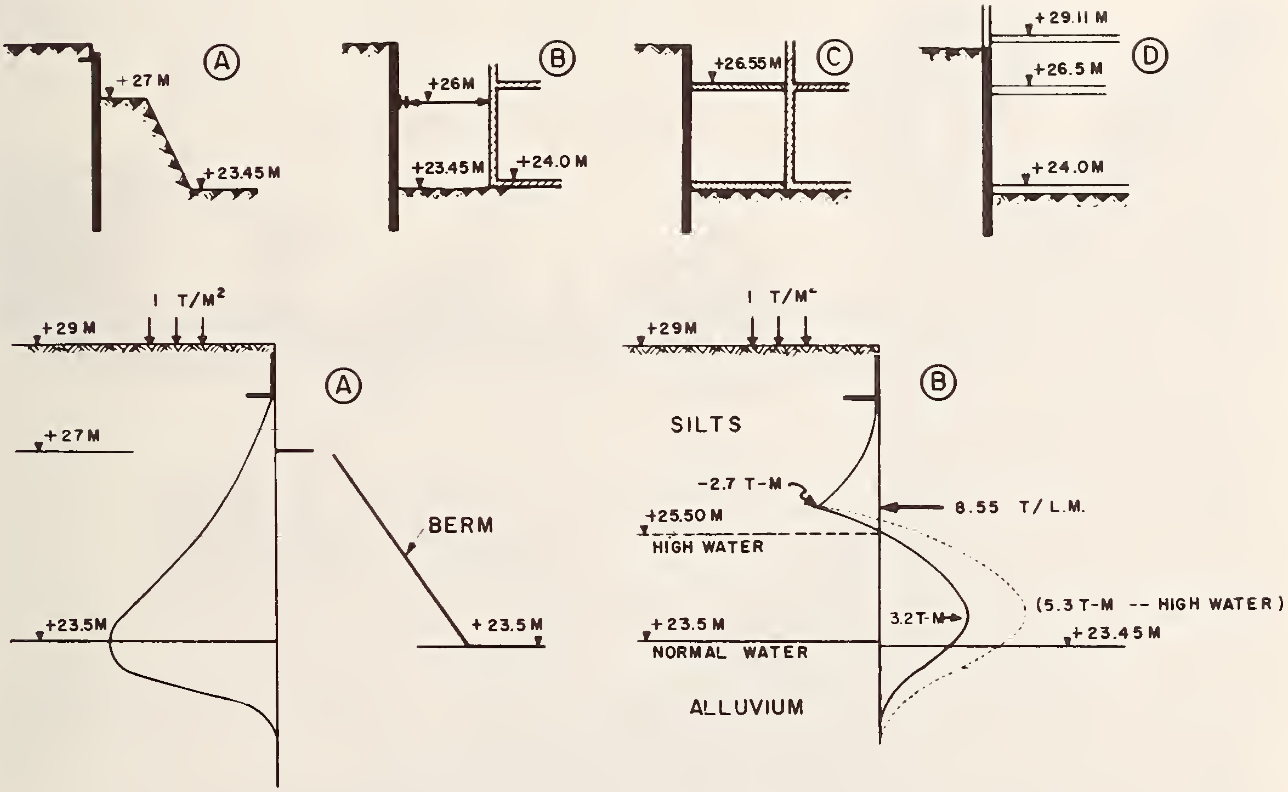
SURESNES-"CENTRE CEGOS" TYPICAL SECTION

FIGURE 14



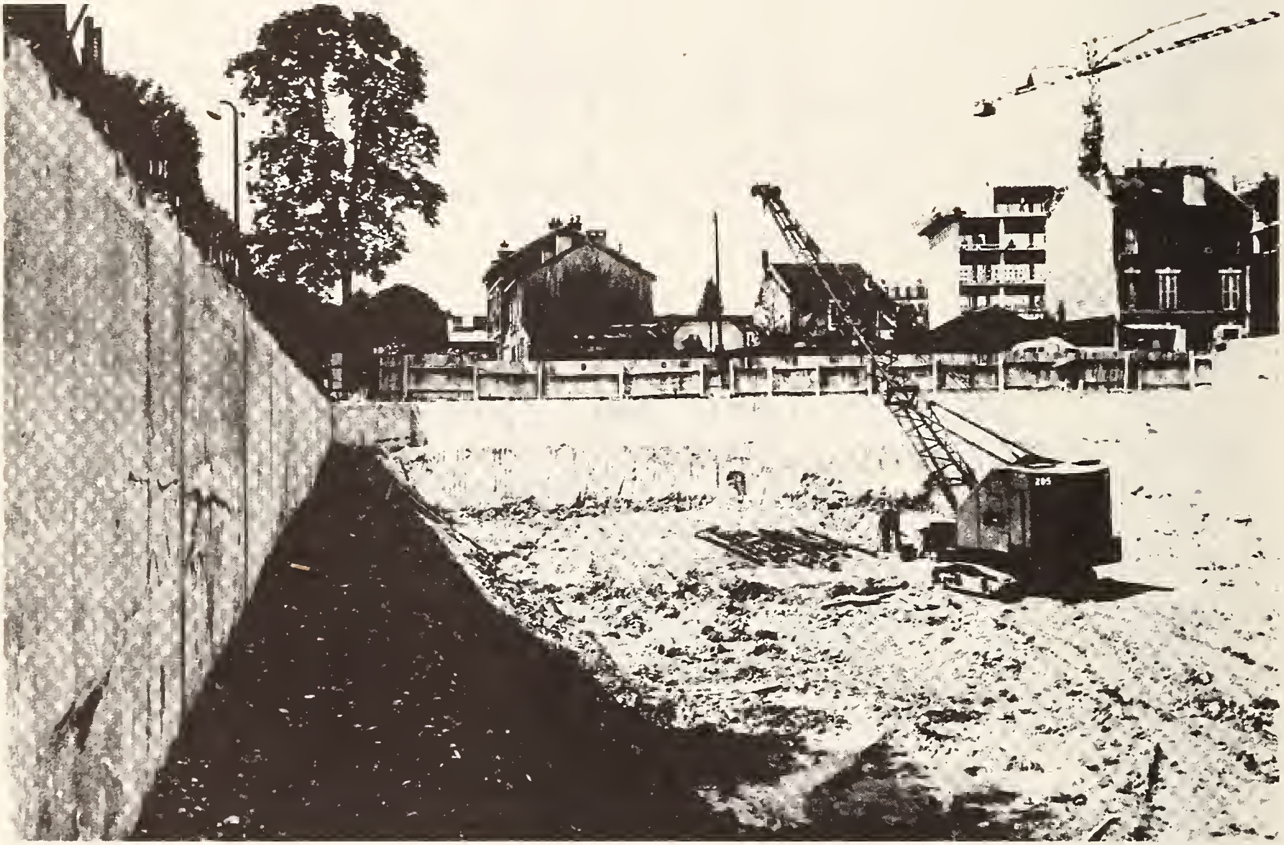
ILE DE LA JATTE PANOSOL WALL
TYPICAL SECTION.

FIGURE 15



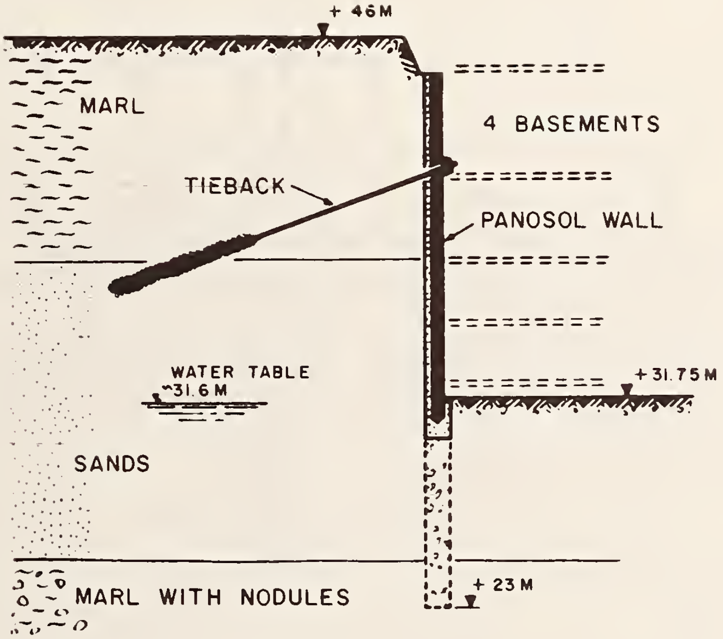
NEUILLY-ILE DE LA JATTE--CONSTRUCTION METHOD AND MOMENT DIAGRAM

FIGURE 16



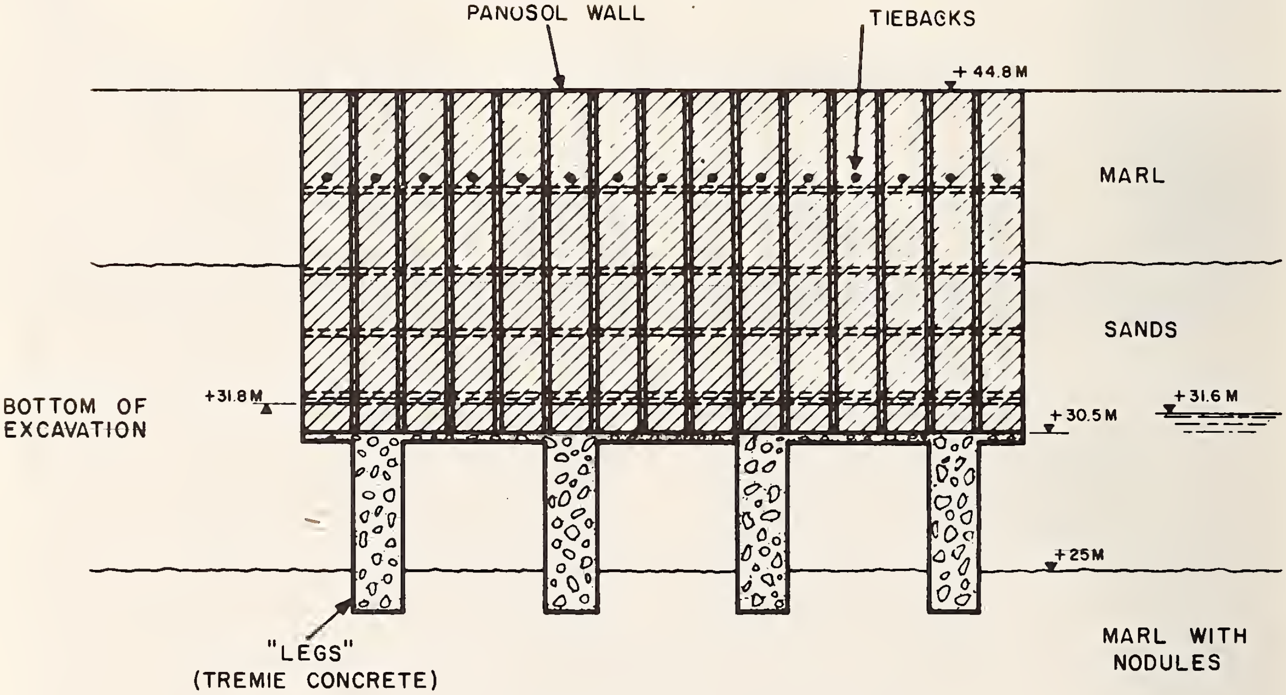
VIEW OF PANOSOL WALL
AT Ile de la JATTE IN NEUILLY

FIGURE 17



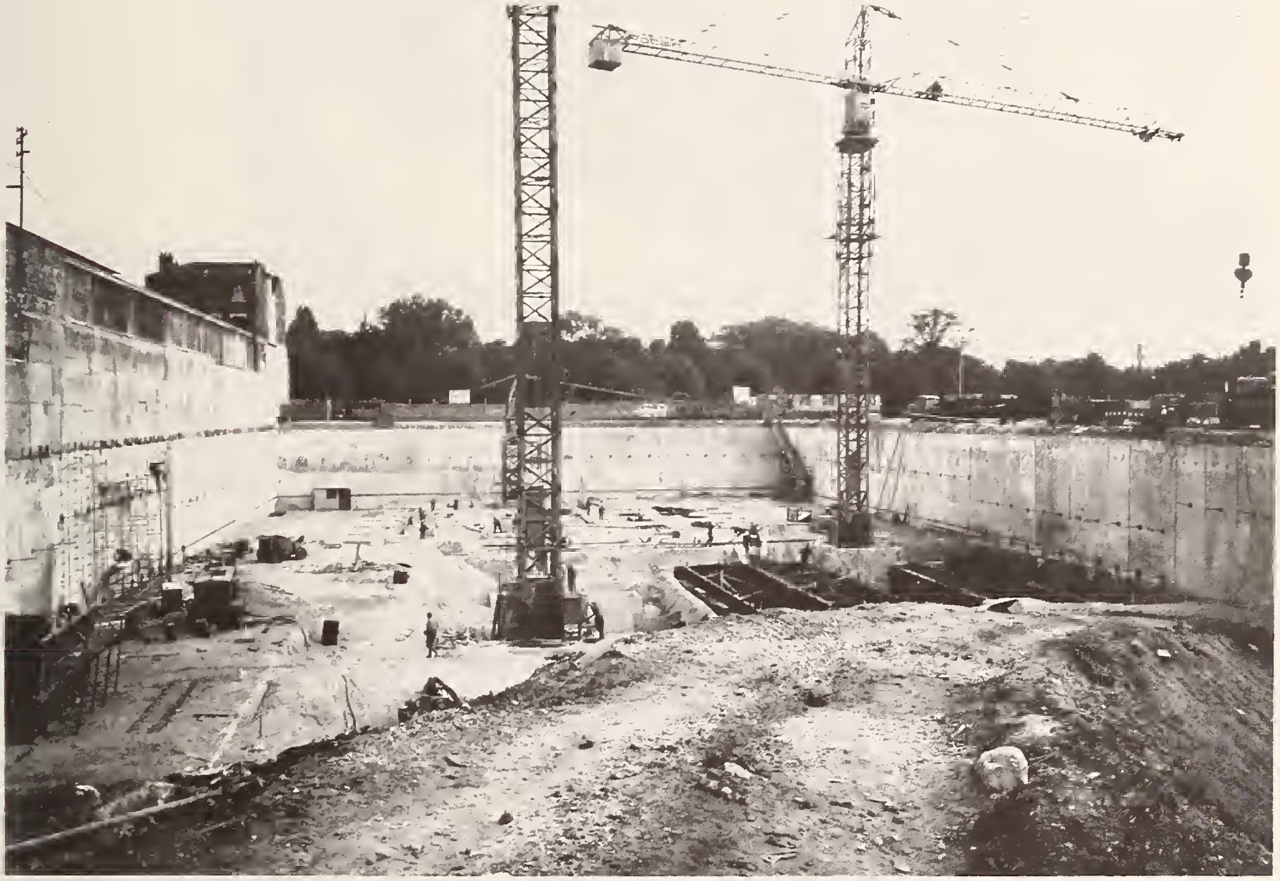
TYPICAL SECTION PANOSOL WALL
COURCELLES – ST. HONORÉ

FIGURE 18



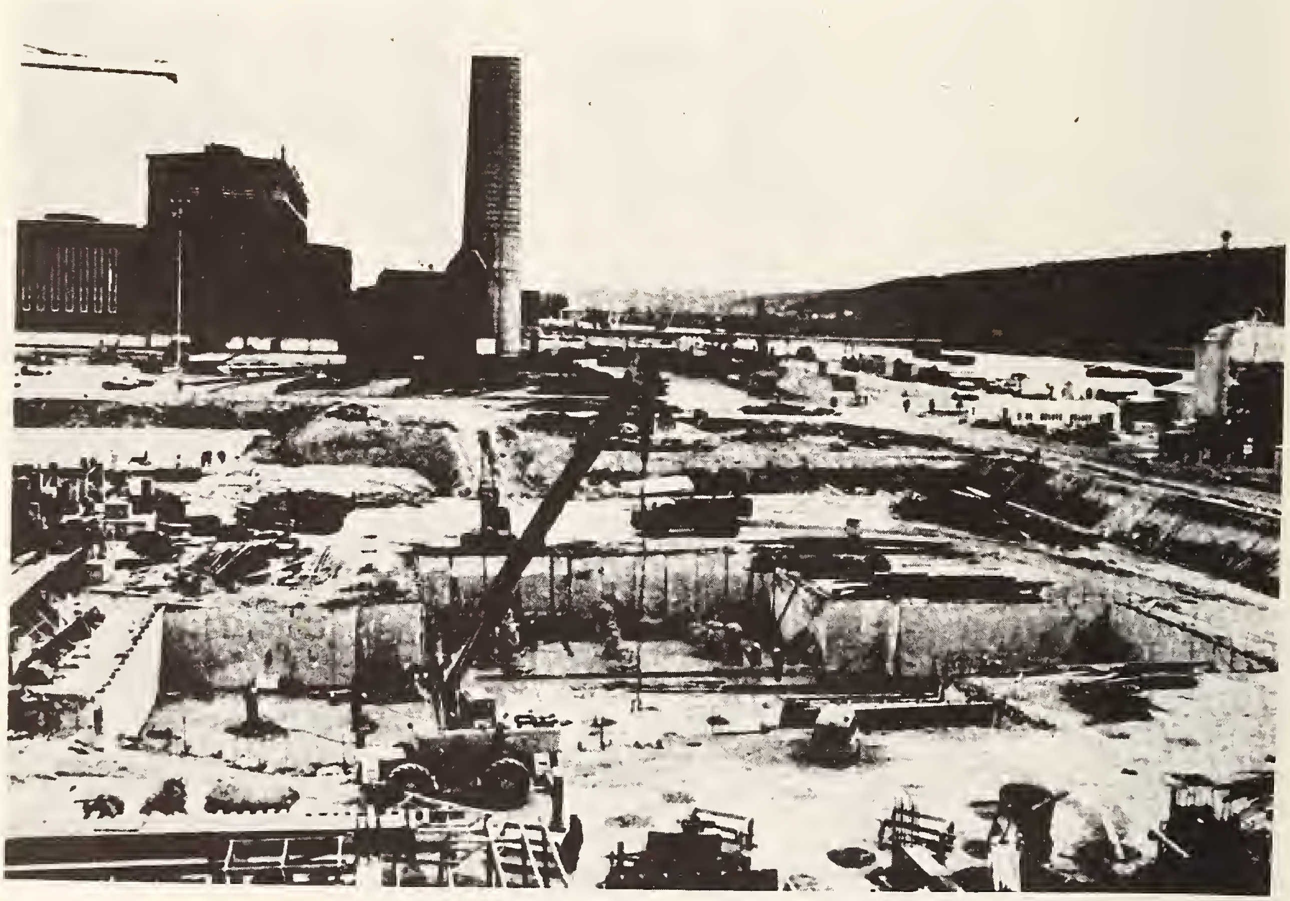
PARIS-- COURCELLES - ST. HONORÉ TYPICAL ELEVATION

FIGURE 19



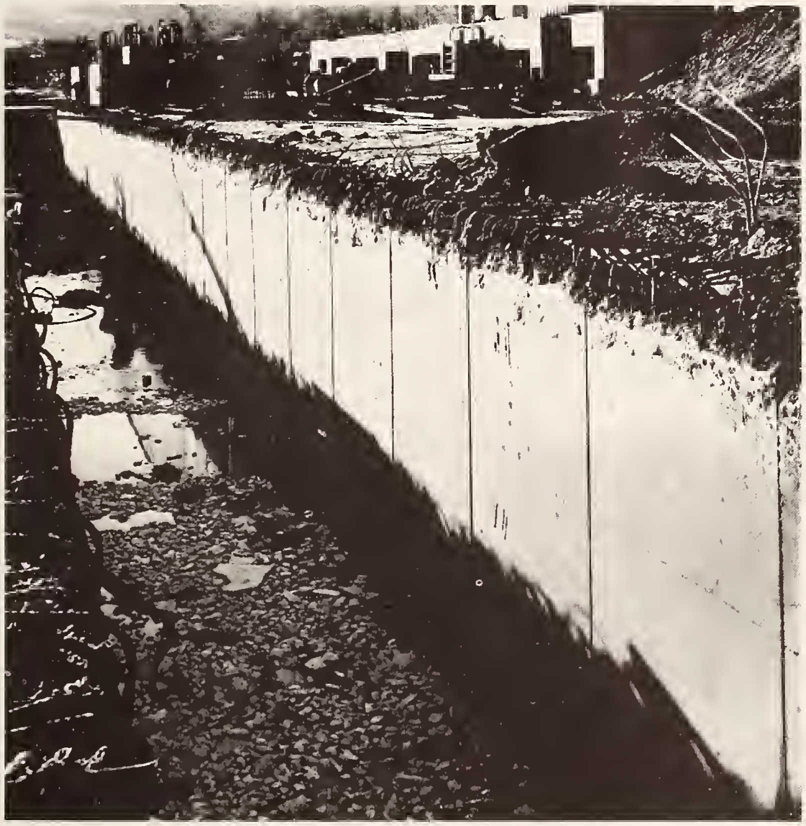
VIEW OF PANOSOL SITE
AT ST. HONORE IN PARIS

FIGURE 20



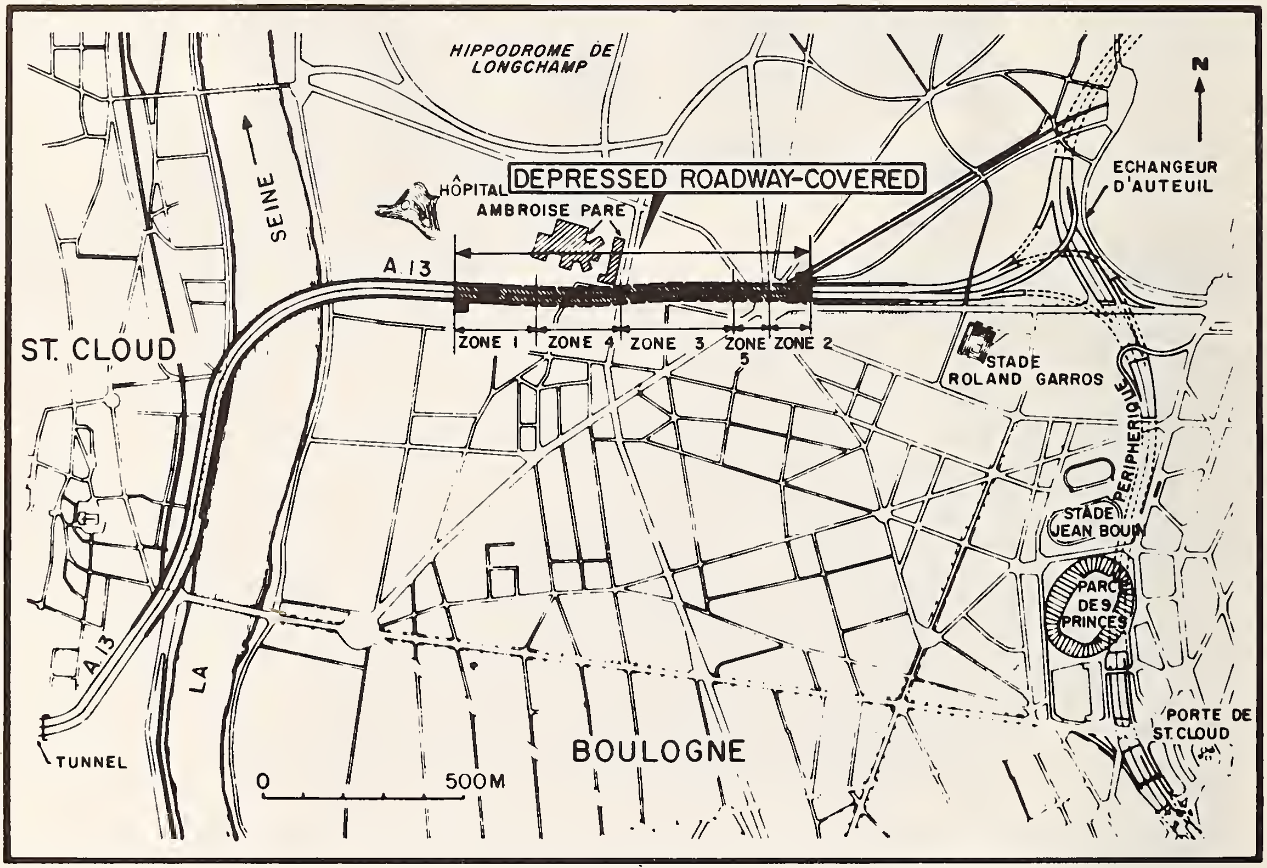
PANOSOL WALLS AT
PORCHEVILLE POWER PLANT

FIGURE 21



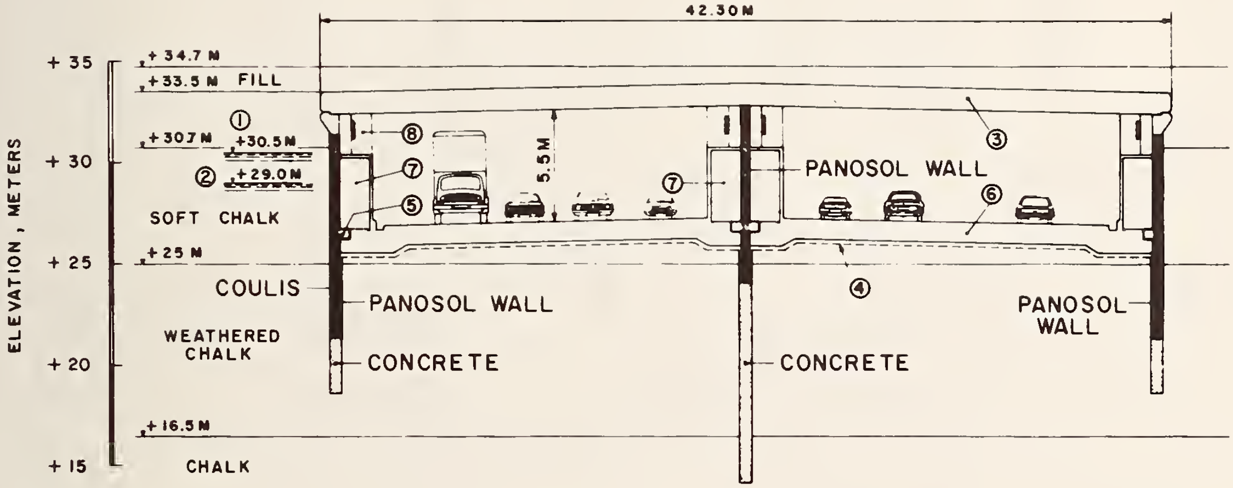
VIEW OF PANOSOL WALL
FOR OUTFALL CANAL AT
PORCHEVILLE POWER PLANT

FIGURE 22



EXTENSION OF HIGHWAY A13--PLAN.

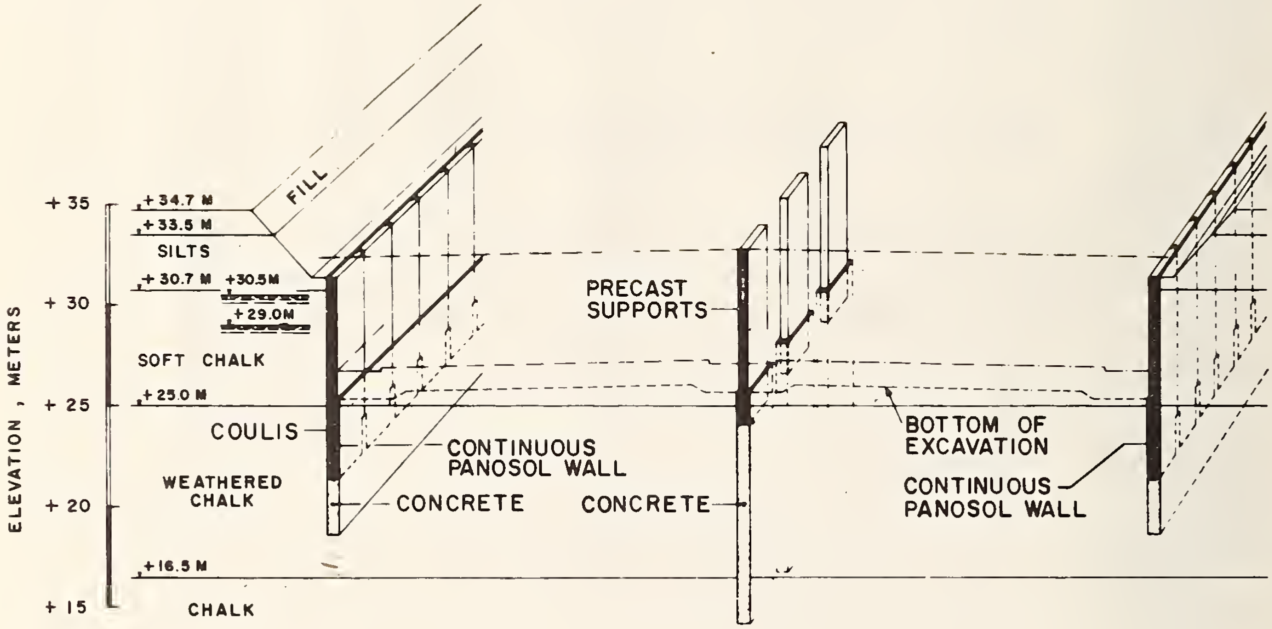
FIGURE 23



① DESIGN WATER TABLE
② CONSTRUCTION WATER TABLE
③ CONCRETE SLAB
④ BOTTOM OF EXCAVATION
⑤ STRUCTURAL KEY
⑥ BASE MAT
⑦ VENTILATION GALLERY
⑧ LIGHTING GALLERY

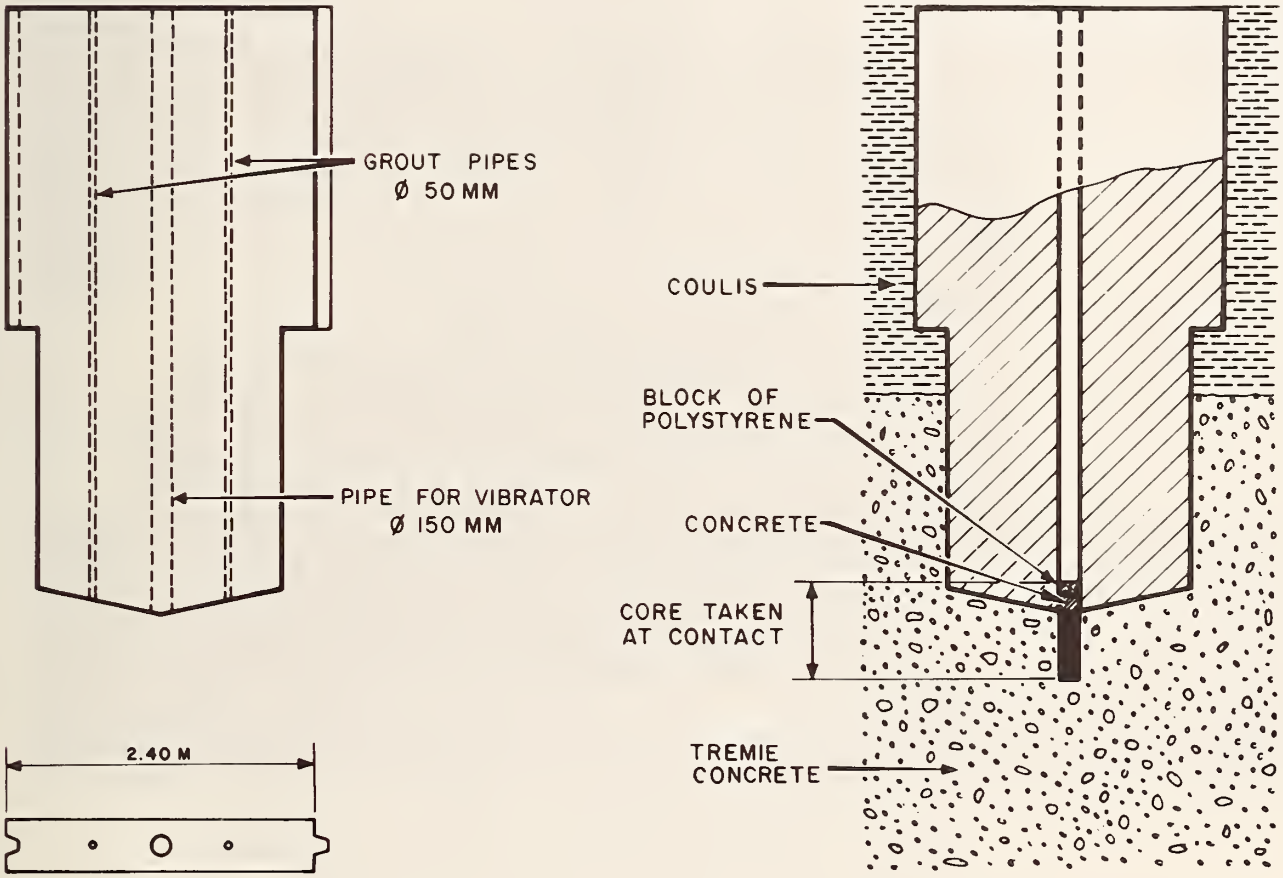
EXTENSION OF HIGHWAY A13. TYPICAL SECTION THROUGH THE DEPRESSED ROADWAY.

FIGURE 24



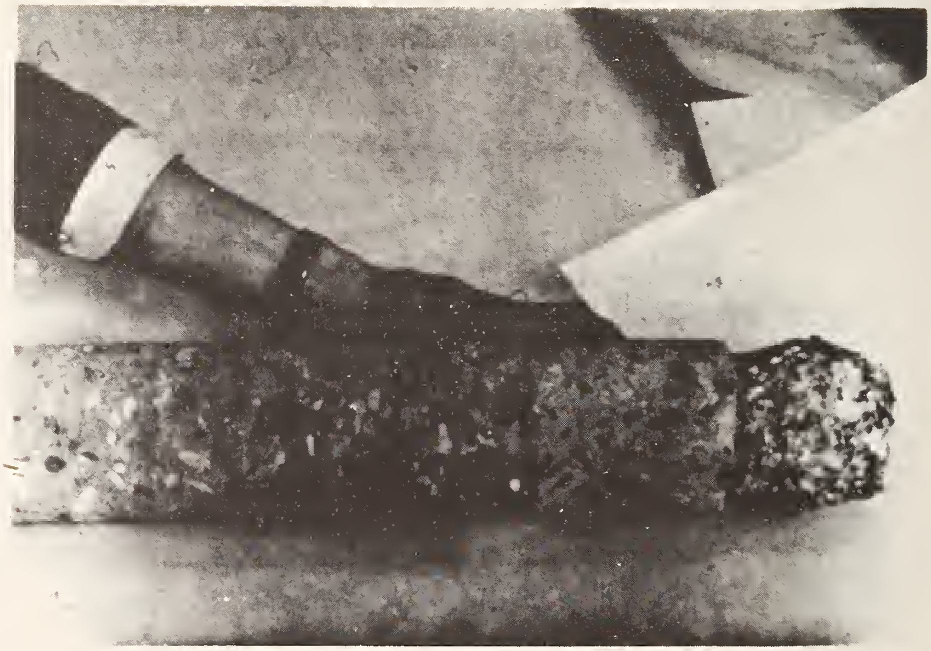
EXTENSION OF HIGHWAY A13 PERSPECTIVE VIEW OF THE DEPRESSED ROADWAY.

FIGURE 25



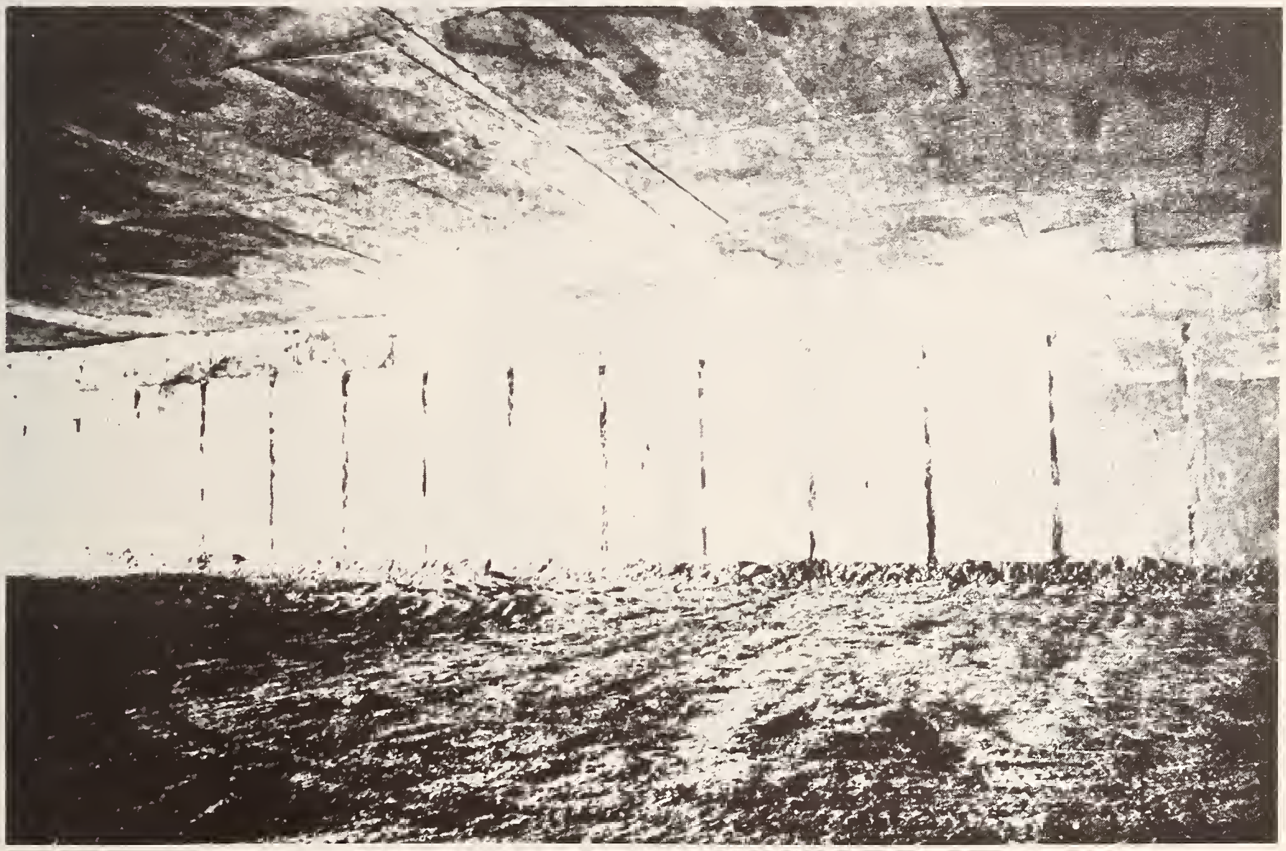
EXTENSION OF HIGHWAY A13 DETAILS OF THE FIRST PANELS

FIGURE 26



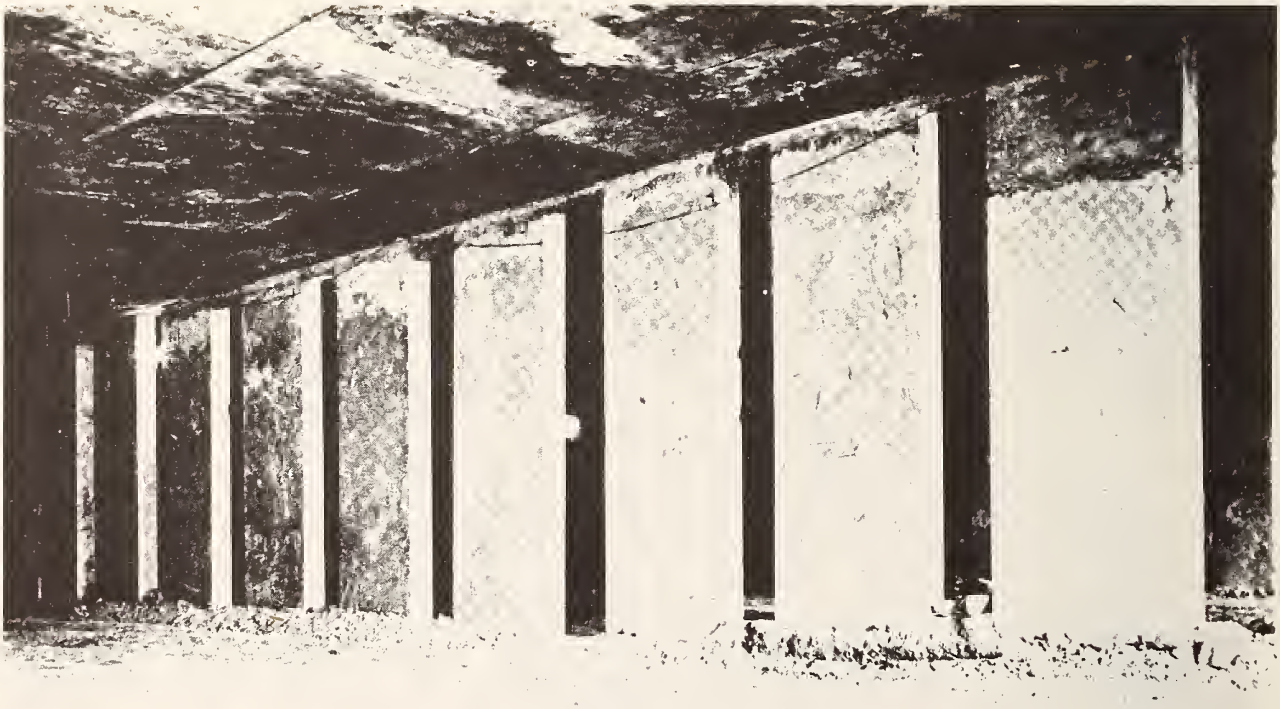
CONTACT BETWEEN PANOSOL AND
TREWIE CONCRETE AT A-13 SITE IN PARIS

FIGURE 27



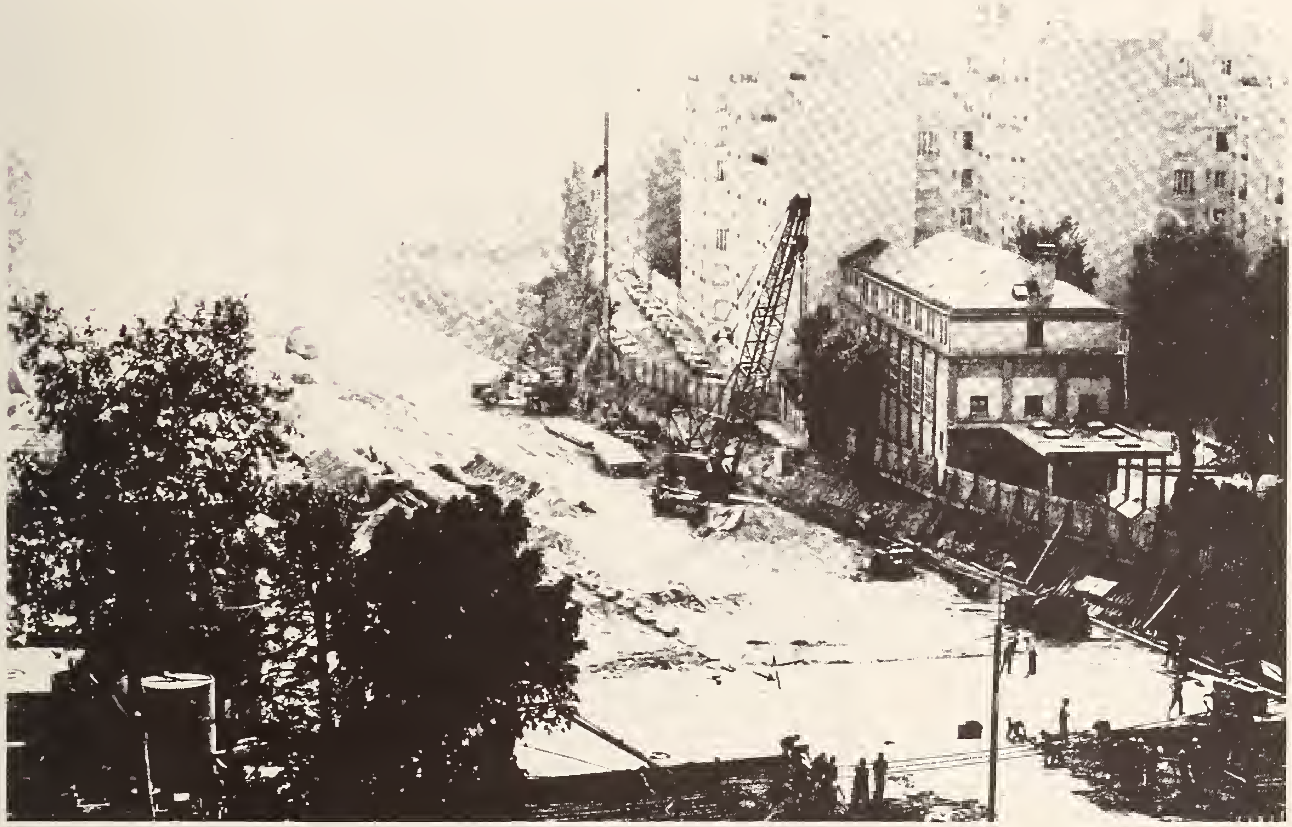
LATERAL PANOSOL WALLS BEFORE JOINT TREATMENT AT A-13 SITE IN PARIS

FIGURE 28



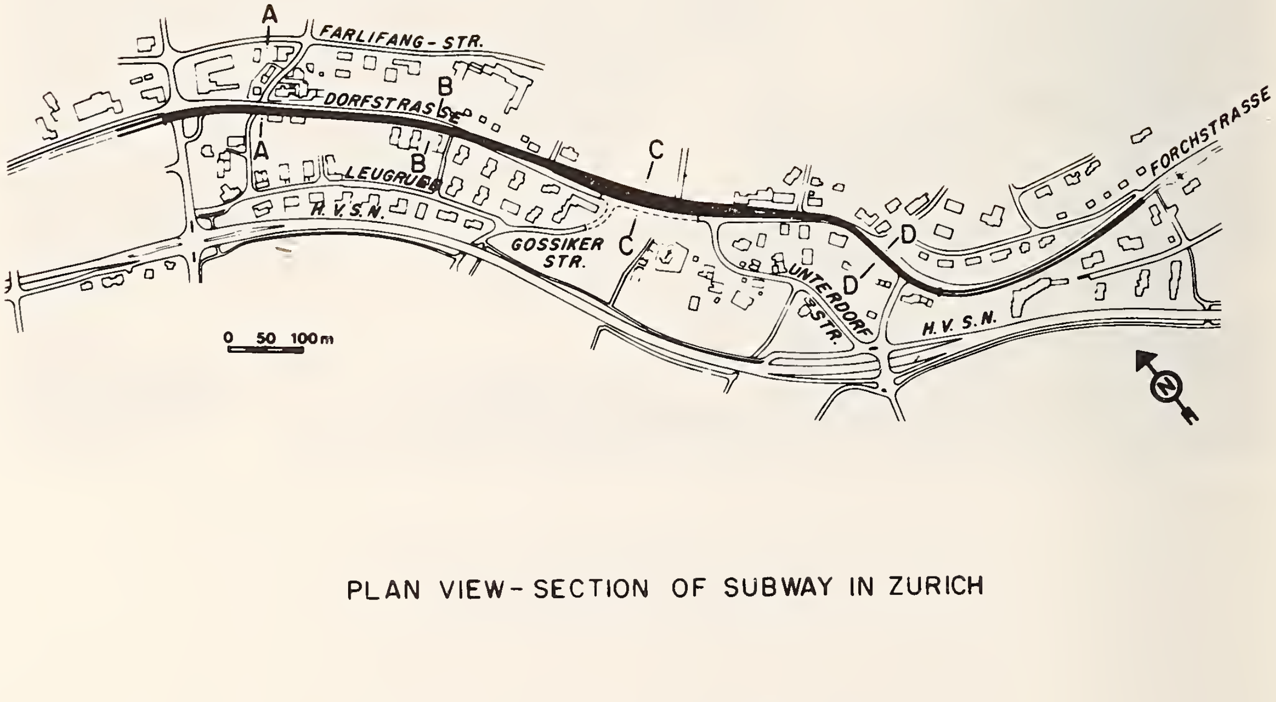
CENTRAL PANOSOL PANELS AT
A13 SITE IN PARIS

FIGURE 29



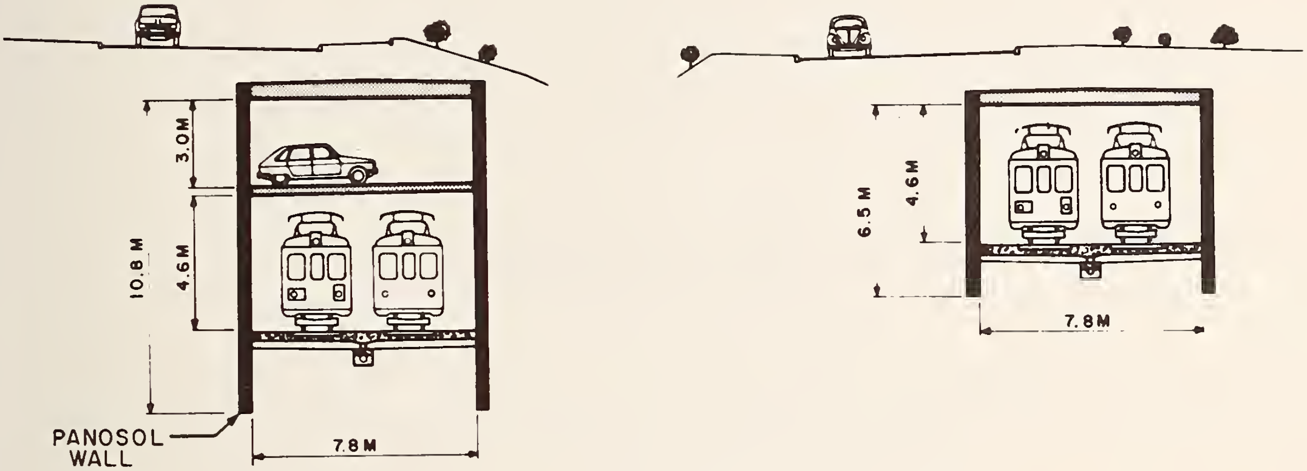
A 13 CONSTRUCTION SITE

FIGURE 30



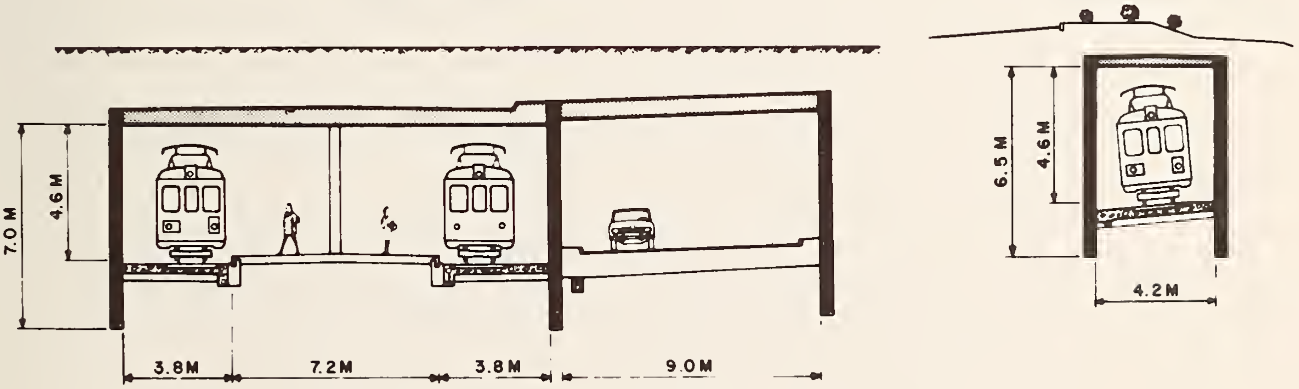
PLAN VIEW - SECTION OF SUBWAY IN ZURICH

FIGURE 31



SECTION A-A

SECTION B-B



SECTION C-C

SECTION D-D

SECTIONS THROUGH ZURICH SUBWAY

FIGURE 32

V. GEOTECHNICAL PERSPECTIVES ON UNDERGROUND METRO SYSTEMS

By

David J. D'Appolonia
Principal, E. D'Appolonia Consulting Engineers, Inc.
Pittsburgh, Pennsylvania, U.S.A.

Symposio Panamericano Sobre Transito Rapido METRO
Caracas, 23-27 October 1972

GEOTECHNICAL PERSPECTIVES ON UNDERGROUND METRO SYSTEMS

INTRODUCTION

Geotechnical considerations play an important role in most every civil engineering aspect of the planning, design and construction of underground tubes and stations for urban mass transportation systems. Major planning decisions involving the alignment and grade of the Metro and the type of construction, whether tunnel or cut-and-cover, are strongly influenced by geotechnical factors. Soil and groundwater conditions have a decisive influence on the type and design of the structural support systems, the requirements for protecting adjacent utilities and buildings, the methods used for construction and, in many instances, the format of the construction specifications and contract documents. In other words, the feasibility and cost of Metro construction are controlled to a significant extent by the character of the subsurface materials.

The geotechnical engineering problems associated with Metro construction are complex due to the influence of the many interrelated technical factors and because of the influence of difficult-to-control contractor behavior. Moreover, the subsoil conditions in many metropolitan areas are poor. Most major cities are near navigable waterways where the underlying soils are often weak and compressible, and the groundwater level is near the surface.

This paper discusses, from a geotechnical point of view, some of the important factors that should be considered in the planning, design and construction of Metro systems. The interrelationship between design and construction considerations and the consequent need for an integrated approach to engineering and construction is discussed. Potential damage to adjacent streets, utilities and buildings caused by movements associated with tunnel or cut-and-cover construction is often a most important design criterion in selecting the alignment and grade of the Metro and the type and method of its construction. Therefore, particular emphasis is given to the problems of predicting movements and their effects on adjacent structures.

GEOTECHNICAL ENGINEERING REQUIREMENTS

From the point of view of the geotechnical engineer, there are three main engineering requirements for the design and construction of an underground Metro system. The first and foremost requirement is that an alignment, grade and method of construction should be chosen such that it will be feasible to build the transportation tubes and stations economically. The cost of construction depends strongly on the subsurface conditions. Rational planning and design decisions, and elimination of excessive construction time and cost overruns are in large measure dependent on the ability of the geotechnical engineer to interpret correctly the in-situ soil and groundwater conditions and to anticipate at an early stage the type and consequences of construction difficulties that will be encountered during the work. The second major concern of the geotechnical engineer involves the consequences of construction in terms of movements causing damage and disruption to nearby streets, utilities and buildings. Decisions on the appropriate type of construction, the conception and details of the temporary support system, and the methods of excavation and groundwater control should be made only after thorough analysis of the movements associated with the various alternative construction procedures and the consequent needs and costs of underpinning or otherwise protecting adjacent structures. In unfavorable circumstances, the cost of underpinning can approach or exceed the cost of tunnel or cut-and-cover construction itself. The third concern of the geotechnical engineer is to determine the loads acting on the permanent structure, including not only the loads due to soil weight, surcharge and water pressure, but also loads due to other influences such as earthquakes and future adjacent construction.

There are many components to the total cost of excavating a tunnel or open cut. The cost of the obvious components--soil removal and the support system--can be less important than components such as protecting adjacent structures and utilities, relocating utilities, controlling groundwater and furnishing temporary access to adjacent facilities. Moreover, the engineer can ill afford to take chances with the safety of an excavation. The major cause of worker death on construction is "excavation cave-ins."

Determining feasible and economical methods of construction, while an obvious requirement, is not always readily satisfied in underground works. The decision of whether to construct using tunnel or cut-and-cover is often the most important step in the design. Considerable experience is available for judging the feasibility and cost of tunneling in various types of soil formations and groundwater conditions. The feasibility and economics of cut-and-cover construction depend primarily on the required depth of excavation.

However, uncertainties with respect to the precise nature of the soil and groundwater conditions that will be encountered introduce major difficulties in applying past experience and empirical data. For example, tunnel construction is particularly susceptible to difficulties resulting from stratigraphic details that are difficult to predict prior to actual excavation.

Moreover, the advent of modern tunneling machines with their high construction speed has created the need to predict probable ground conditions far more accurately than was perhaps needed when slower and more flexible hand-mining techniques were employed. It does not require much unexpected overbreak in a tunnel, a higher water table and soft inverts, high seepage gradients in minor sand or silt lenses, or a miscalculation of material hardness and abrasiveness to slow construction progress, create problems of loss of ground and significantly escalate the cost of a project. Therefore, a thorough exploration, performed with the aim to obtaining information for construction as well as design, is essential in most circumstances.

The effect of construction on nearby streets, utilities and buildings is another dominant consideration that may severely limit the available choices of suitable construction and, in some instances, may govern the choice between tunnel and cut-and-cover. Design of an underground transportation facility in an urbanized area cannot be completed without a thorough analysis of the requirements for protecting adjacent structures. Underpinning requirements and costs should enter directly into the initial decisions on location, grade and type of construction. If potential problems resulting in damage to adjacent facilities are not properly assessed, then expensive law suits, cost overruns and construction delays will inevitably result. Experience has repeatedly shown that large damage costs and construction difficulties are most often associated with unanticipated events that could have been avoided by better planning, design or construction.

The problem of determining the loads acting on the temporary and permanent support systems is relatively well defined and straightforward. Loads have been measured in many tunnel and braced excavation projects and proven semi-empirical design procedures have been developed from the observational data. In the case of cut-and-cover construction, there is a large body of empirical evidence relating magnitude and distribution of load on the lateral support system to the soil type. In the past few years, design criteria for tunnel liners have been improved so that the strength mobilized in the surrounding soil is properly considered. Peck (1969) and Peck, et al. (1972) present excellent summaries of the state of the art for designing support systems for braced excavations and soft ground tunnels.

INTERRELATIONSHIP BETWEEN DESIGN AND CONSTRUCTION

Design and construction are perhaps more interdependent and inseparable on underground construction works than in any other area of civil engineering construction. The close relationship is the result of the dependence of behavior--i.e., loads and movements--on construction procedures and details. Further dependencies result from the intrinsically heterogeneous nature of natural soil deposits whose geological details cannot be known prior to actual full-scale observation during the construction process. Ground movements are especially influenced by factors that are very difficult to predict correctly, such as geological details, construction details, and workmanship. Therefore, in most cases, engineering analyses and predictions must be based almost entirely on observational data and prior experience. Regrettably, field measurements of movement caused by underground construction have been made infrequently in the past and observational data are often not sufficiently complete to formulate generally applicable empirical rules. However, it is now becoming standard practice on Metro projects to include field measuring devices on and near adjacent structures so that designs and construction procedures can be evaluated and, if necessary, modified during construction.

The above considerations, among others, have pointed out the need to establish contractual frameworks conducive to the integration of design and construction. As a result, the traditional formats of contract documents and specifications are being scrutinized and altered as experience increases with underground construction in urban areas.

By long tradition in civil engineering construction, contract documents deal almost exclusively with the structural features of the completed facility. The engineer who designs the facility does not specify how the construction is to proceed. Therefore, in underground works, decisions on the method of executing the work, temporary support systems, groundwater lowering and control, and protection of adjacent structures during construction are usually left to the option of the contractor. The design engineer often declines to take any responsibility for the temporary support and movements of adjacent ground, so the responsibility is passed on to the contractor.

There are both advantages and drawbacks to following these procedures. The great advantage is that the contractor is free to make his own schedule and use his ingenuity and expertise in construction methods to complete the project in the most economical manner possible. However, experience is demonstrating that in many cases the drawbacks far outweigh this advantage. Loads and especially movements depend on the method and details of construction. It is now recognized that traditional contract systems leave some of the most essential elements of design to the contractor. Consequently, trends are developing toward increasing engineering responsibilities, primarily by using more comprehensive and detailed specifications on the method of construction. There is also increasing use of realistic performance-type specifications. It is now common to provide the contractor with parameters for designing temporary support systems and criteria for underpinning or otherwise protecting adjacent structures.

Another major geotechnical consideration in formulating the contract documents involves the arrangements for equitably apportioning risks associated with unanticipated underground conditions and soil behavior. Significant cost increases can result from litigation and lost time directly attributable to provisions, or lack of provisions, for handling "changed conditions" resulting from interpreted subsurface information. Several especially important points, from the contractor's point of view, that tend to create problems and add contingencies are: (1) the use of arbitration procedures whereby the owner or engineer is the sole arbitrator of all disputes; (2) clauses requiring the contractor to assume the liabilities of the owner with respect to site conditions and engineer interpretations of subsurface conditions; and (3) use of the term "as directed by the engineer" without clearly stating what is required, especially in terms of performance. Serious effort must be given to making contracts that are fair to all of the involved parties and that are capable of resolving disputes expediciously and economically.

Some of the evolving contracting methods being used to overcome these problems involve the more extensive use of performance specifications, the construction manager approach, where design and construction are jointly managed by the construction manager, and the use of negotiated incentive contracts that allow for engineering changes during construction. All of these methods are aimed at integrating design and construction, providing the contractor with the best possible information for establishing construction methods and costs, applying the risks associated with the site conditions to the owner and the responsibility for good construction techniques to the contractor, and finally reducing the spiraling costs of underground construction associated with changed conditions and damage claims.

THE PROBLEM OF MOVEMENTS

Considerable experience and judgment is required to reach a satisfactory conclusion on the potential for damages and the need for underpinning and protection of adjacent facilities. Depending on the nature of the nearby structures and the costs and risks involved, it is sometimes best to underpin or to purchase and demolish. In other circumstances, it may be most economical to allow damage and later repair it; while in some cases, it is best to modify the design or construction method to prevent damage. Regardless, rational evaluation of the available alternatives requires that the engineer know the allowable movement of the nearby structures and the magnitude and distribution of movement associated with the various feasible alternatives. Most significantly, the engineer must make a judgment of the movements that might result from minor geological deviations and occasional departures from optimal construction techniques and workmanship.

Depending on the results of his analysis, as well as legal and insurance considerations, the engineer must then decide how to specify requirements for movement control. The engineer may decide to design the necessary underpinning and protection measures, he may specify criteria for designing the underpinning and protection measures--leaving the actual design to the contractor's option, he may write performance specifications with respect to allowable movements or he may leave the entire problem to the option of the contractor and only require that construction plans are subject to the engineer's approval. However, irrespective of what form of contract is established, the engineer must carefully consider the problem of movements. He must have on hand the necessary background and analysis so that he can make adequate decisions in approving contractor techniques and methods and so that decisions to modify construction can be taken if problems develop. In this regard, the engineer should plan and execute a system of measurements and observations for controlling the construction and judging contractor performance.

Movements Causing Damage

The first step in a rational analysis of the movement problem is to make a study of the buildings and utilities along the metro alignment and establish ranges of allowable movement. In most cases, the movements caused by tunnel or cut-and-cover construction that an existing building can safely tolerate are less than the movements that the same structure can experience without damage during settlement under its own weight. Settlement caused by Metro construction usually occurs rapidly and produces differential settlement in directions both perpendicular and parallel to the alignment.

The subject of allowable movements is very complex and much engineering judgment needs to be exercised to reach a sound conclusion in each individual case. The factors listed in Table 1 should be considered when making these judgments.

TABLE 1 - FACTORS INFLUENCING TOLERABLE ADDITIONAL MOVEMENT OF EXISTING BUILDINGS

Type of Movement
Rate of Movement
Magnitude and Distribution of Movement
Type and Construction of Building
Age and Existing Condition of Building

D'Appolonia (1971) discusses how each of these variables influences behavior of buildings when subjected to movements caused by nearby construction.

If existing buildings are in good condition and additional settlement is limited to less than about 2 or 3 cm, serious damage will be avoided in most all cases. There are certain cases where considerably more additional movement can be safely tolerated. However, simple generalities tend to be untrustworthy and each case needs to be studied individually to make an accurate prediction.

Another important factor to consider is the degree of damage that might be acceptable in comparison to the cost of using expensive or time-consuming construction procedures to prevent the damage. Small settlement of a building, causing minor cracking and architectural damage but involving no risk of structural damage, can sometimes be allowed when the cost of repairing the damage is much less than the cost of preventing it.

Movements Caused by Cut-and-Cover Construction

A large number of interrelated factors influence the magnitude and distribution of movements accompanying cut-and-cover construction. Movements depend on the dimensions, and especially the depth, of the excavation; the soil conditions; the rigidity and the method and sequence of installing the support system; the time that the excavation is left open; and the construction details and workmanship. The method of groundwater control may also have an important influence on movement. Due to these complexities, accurate quantitative predictions of soil movement cannot be made on the basis of soil testing and theoretical analysis alone. Evaluated field measurements are the essential key for making predictions and guiding judgment. Peck (1969) and D'Appolonia (1971) summarize available observational data and make recommendations for estimating movements associated with braced excavations.

Excavation removes a mass of soil and water and produces a reduction in total stress along the sides and bottom of the cut. In many cases, the groundwater is lowered to facilitate the removal of soil and construction inside the excavation. The reduction in total stress during excavation causes the soil to move inward toward the excavation and upward at the bottom of the cut. The upward movement of the bottom is accompanied by an inward movement of the soil below excavation level. As these movements occur, the soil adjacent to the excavation undergoes lateral displacement and settlement. Figure 1 illustrates the relationship among ground settlement, lateral movement and bottom heave for a sheeted cut-and-cover excavation in soft to medium clay. The volume of settlement of the adjacent ground was approximately equal to the volume of inward movement of the sheeting. Sheeting movements were related to the heave measured below excavation level.

Installation of lateral bracing is always preceded by excavation to the brace level. Thus, in studying the relative influence of the many factors that control overall behavior, it is useful to differentiate between wall movements occurring below the excavation level and wall movements occurring above the excavation level.

Once excavation has passed a given elevation, further inward movement above that elevation is controlled entirely by the procedures and details of brace installation. Some of the important factors controlling these movements are listed in Table 2.

TABLE 2 - FACTORS CONTROLLING LATERAL WALL MOVEMENTS OCCURRING ABOVE EXCAVATION LEVEL

Horizontal and Vertical Brace Spacing
Depth of Excavation Below Brace Level Before Brace Is Installed
Length of Excavation Parallel to Wall Made at Any One Level Prior to Installing Braces at That Level
Elapsed Time Between Excavation and Brace Installation
Details of Prestressing and Wedging Braces
Details of Excavating and Placing Lagging Between Soldier Beams

Experience has shown that unless careful attention is given to the excavation and bracing details, avoidable lateral wall movements occurring above the excavation level will contribute a large percentage to the total lateral movement and ground settlement outside the excavation. Moreover, movements associated with poor excavation and bracing details are usually irregular and lead to damaging differential settlement.

Movements occurring below excavation level are determined by the combined resistance to inward movement provided by the embedded portion of the wall and the soil below the base of the cut. In certain cases, both factors may be important. However, for typical soldier pile or steel sheeting construction, the magnitude of the movements depends chiefly on the properties of the soil. Figure 2 summarizes maximum movements measured during excavation in various types of soil. Different symbols are used for cuts supported using standard soldier piles or steel sheeting and cuts supported by more rigid cast-in-place concrete walls constructed using the slurry trench method.

The largest movements and therefore the most serious problems are associated with cut-and-cover excavations in soft and medium clay or compressible silt. For major excavations in these soils, maximum ground settlements are commonly on the order of 1 to 2% of the excavation depth. For excavation in soft and medium clay, lateral wall movements are associated with settlements of about equal volume. Measurements on well-designed and constructed sheeted excavations show that 60 to 80% of the total lateral wall deflection takes place below excavation level. These movements depend on the passive resistance that can be mobilized by the soil immediately below the base of the cut. This passive resistance is related to the bottom stability number N which is the ratio of the overburden stress at the base of the cut to the undrained shear strength of the clay. When excavation reaches a depth where bottom stability failure is approached ($N \approx 6$), both the lateral movement and the ground settlement increases sharply. Both the magnitude of the movements and their distribution as a function of distance from the cut depend on bottom stability. In soft clays where N is greater than about 6, settlements of up to 0.2% of the excavation depth may be experienced at distances of 3 or 4 times the depth of excavation. Peck (1969) prepared the diagram reproduced in Figure 3 for estimating the settlements that might be expected under various conditions.

While the bending resistance of steel sheeting is not adequate to restrict lateral wall movements below excavation level when bottom stability failure is approached, there is an increasing body of evidence to suggest that stiff cast-in-place concrete walls constructed by the slurry trench method can reduce movements, even in soft clays, by a factor of 2 to 5 compared with steel sheeting. Other methods involving soil freezing, use of air pressure and excavating by dredging underwater have also been effective in reducing movements. However, these methods are often impractical or the cost is prohibitive.

Well-constructed excavations in granular soil or stiff clay usually cause maximum movements on the order of only 0.1 to 0.2% of the excavation depth. Movements will generally be confined within a horizontal distance equal to the excavation depth. However, large and erratic settlements can accompany excavation below groundwater level in cohesionless soils due to failure to control groundwater flow adequately. Such cases are not uncommon. Problems are most often caused by careless pumping from sumps leading to loss of bottom stability or by failure to prevent erosion of soil through leaks in retaining walls. Groundwater lowering can also cause consolidation of compressible clays and silts.

Movements Caused by Tunnel Construction

Construction of a soft ground tunnel produces a change in the state of stress in the soil surrounding the tunnel. The stress changes invariably lead to displacements. It is well known that some displacement is necessary to mobilize the strength of the soil and permit advancing the heading. The magnitude of the ground movements is strongly influenced by the depth and size of the tunnel, the method and the details of construction and the workmanship. However, the dominating factors controlling movement are the soil type and groundwater conditions. As is also the case with cut-and-cover construction, evaluated field measurements and observational data are the key for making adequate predictions. Peck (1969) and Peck, et al (1972) summarize available means for estimating movements and make suggestions for minimizing ground movements.

Experience on many tunnel projects indicates that the pattern of movement ordinarily takes the form of a trough-like depression more or less symmetrical about the axis of the tunnel. Figure 4 illustrates typical surface movement patterns associated with driving a single tube. Surface settlements are greatest at points immediately above the axis of the tunnel while horizontal movements are greatest near the inflection point of the settlement trough, which is typically near or beyond the spring line of the tunnel.

Settlement and lateral movements are often directly proportional to the volume of lost ground, i.e., the volume of soil passing across the perimeter of the tunnel cross section. Lost ground may be due to compression or movement of soil at the face of the heading, compression of the support system, and, in shield tunneling, invasion of soil into the tailpiece clearance. Figure 4b shows the settlement of a point on the ground surface directly above a tunnel heading. Curve A is typical of shield tunneling in a well-drained granular soil. The settlement is small until the shield is directly below the point and the rate of settlement is greatest as the tailpiece passes. For this case, the lost ground and corresponding surface settlement are related to the shove. Curve B shows typical movements in a medium clay where a majority of the lost ground is due to plastic flow of the soil across the face. In this case, the rate of settlement of the point on the surface increases as the face is advanced and becomes greatest when the face passes below the point. These characteristic movement patterns result in differential settlements and horizontal movements in directions both parallel and perpendicular to the axis of the tunnel.

In addition to movements caused by lost ground, settlements may also occur due to volume changes caused by the stress changes associated with the tunnelling. Loose sands may undergo volume decrease when subjected to shear stress. Clays may undergo appreciable consolidation due to stress changes or remolding and disturbance of the clay around the tunnel, especially if a shield is used.

The bell shape of the settlement trough compares roughly to the error function or normal probability curve. Use of the curve has no theoretical justification but it does provide an expedient framework for quantifying observational data to provide a means for estimating settlements as a function of distance from the axis of a tunnel. Figure 5a shows the pertinent properties of the error function which are the maximum ordinate of the curve, S_{max}, and the distance, i, from the center of the curve to the inflection points. Figure 5b gives an empirical relationship between i and the tunnel geometry for various soil types and groundwater conditions. The width of the settlement trough tends to be significantly greater, other factors being equal, in plastic clays than in granular materials. Tunneling in sand below groundwater level is the exception. Under these conditions, control of lost ground and movements is most difficult.

Movements associated with tunneling through granular materials are usually small provided that raveling is prevented and the groundwater is completely controlled. In these cases, the volume of lost ground is typically on the order of 0.5 to 1.0 percent of the tunnel cross section. However, if raveling or runs develop, lost ground may increase dramatically, resulting in large and erratic settlement at the ground surface.

In the cohesionless materials, prevention of raveling is extremely difficult and the top, sides and face of the heading must be protected at all times. The greatest problems are encountered when tunneling in granular soils below groundwater level. In these cases, the groundwater level must be lowered and the soils drained to the extent where there are no seepage gradients toward the exposed face. An air pressure of about one atmosphere is often used together with deep well dewatering because complete predrainage of all water-bearing elements is difficult to accomplish. Pregrouting of granular soils, both above and below groundwater level, has been used with great success in a number of cases to control raveling or running ground.

The details of stratigraphy and drainage and the workmanship control the magnitude of settlement associated with tunneling in granular materials. Therefore, the most important task of the designer is to judge the groundwater and raveling conditions that are likely to be encountered and to rule out construction methods that are prone to difficulties in the given site conditions. It is especially important that the engineer maintain strict control over the construction.

Raveling and piping do not occur when tunneling in plastic clay deposits unless granular inclusions are encountered. However, settlements can be large due to plastic flow of the material toward the opening. Movements generally do not occur rapidly so that the heading is lost, but the settlements can be much greater than those above similar well-made tunnels in granular soil.

Air pressure is an effective means of reducing movement because it reduces the stress change due to excavation.

Experience has shown that with good construction techniques, settlement in plastic clays will be small if the ratio $(P_z - P_a)/S_u$ is less than 3 or 4, where P_z is the overburden pressure, P_a is the air pressure and S_u is the undrained shear strength of the clay. Figure 6 gives some data relating $(P_z - P_a)/S_u$ to maximum surface settlement.

SUMMARY

Geotechnical engineering plays a most significant role in planning, design and construction of underground transportation systems. The feasibility and cost of construction are controlled to a large extent by the soil and groundwater conditions. Successful design and construction depend on the ability of the geotechnical engineer to properly interpret in-situ soil and groundwater conditions with respect to the feasibility and cost of cut-and-cover or tunneling and to properly anticipate the effects of minor geologic variations on construction difficulties. All the essential details of the subsurface conditions are difficult to predict prior to actual construction and, therefore, there is a great need to integrate design and construction. A major consideration in selecting alignment and grade and the method of construction involves the consequences of movements associated with construction in terms of damage and disruption to streets, utilities and nearby buildings.

REFERENCES

D'Appolonia, D. J. (1971), "Effects of Foundation Construction on Nearby Structures," Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, State-of-the-Art Volume, San Juan, Puerto Rico.

Peck, R. B. (1969), "Deep Excavation and Tunneling in Soft Ground," Seventh International Conference on Soil Mechanics and Foundation Engineering, State-of-the-Art Volume, Mexico City.

Peck, R. B., A. J. Hendron and B. Mohraz (1972), "State of the Art of Soft Ground Tunneling," Proceedings, North American Rapid Excavation and Tunneling Conference, Chicago.

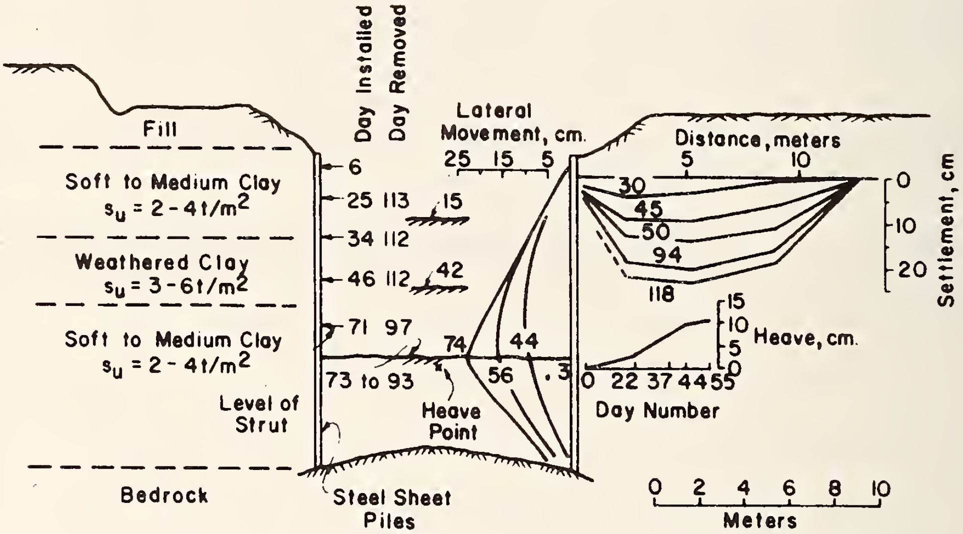
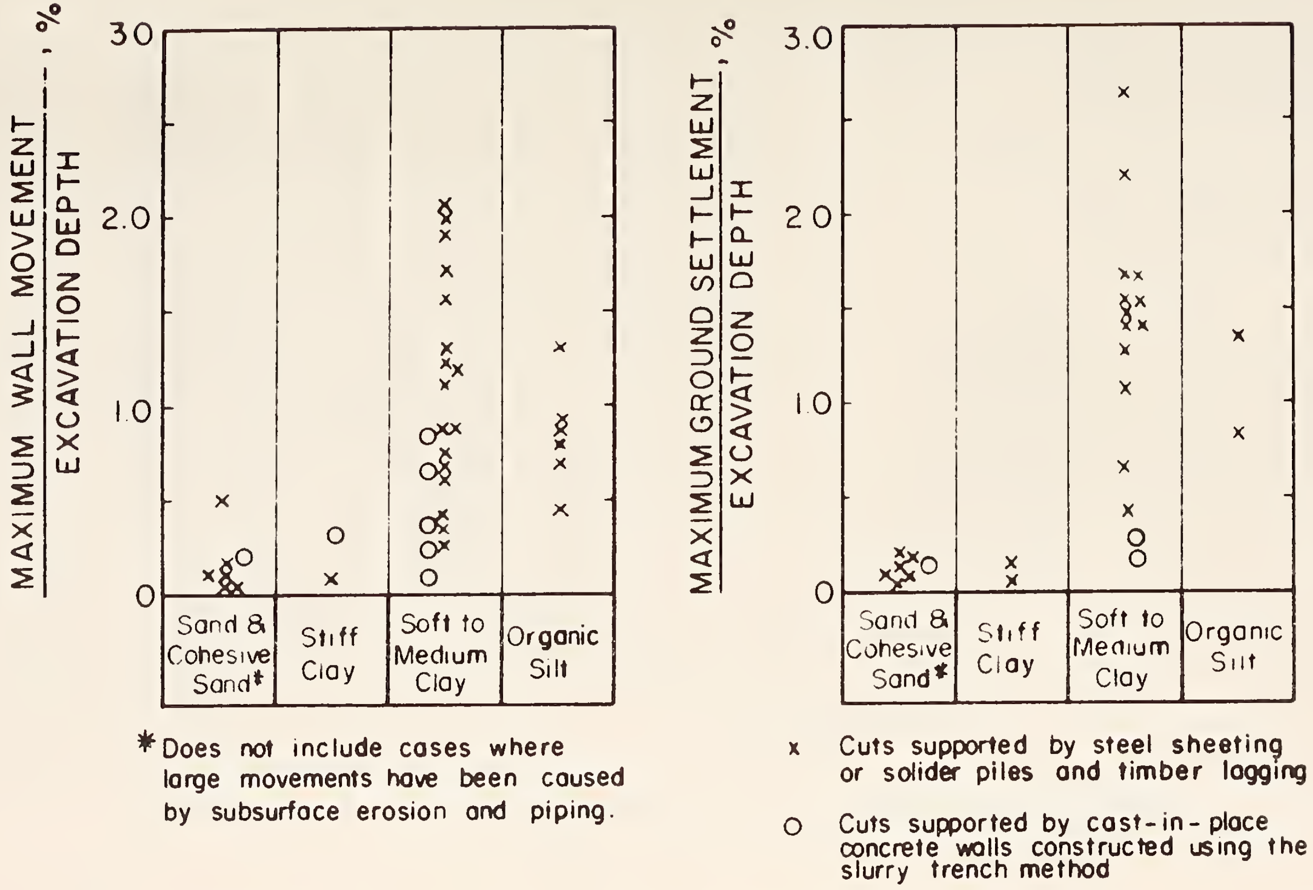


FIGURE I. RELATIONSHIP OF CONSTRUCTION PROGRESS TO MOVEMENTS OF AN OPEN CUT IN SOFT TO MEDIUM CLAY AT VATERLAND IN OSLO

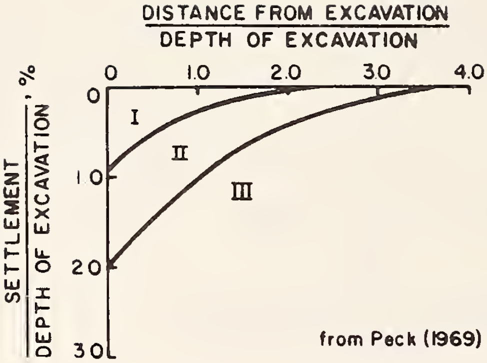


* Does not include cases where large movements have been caused by subsurface erosion and piping.

x Cuts supported by steel sheeting or solider piles and timber lagging

o Cuts supported by cast-in-place concrete walls constructed using the slurry trench method

FIGURE 2 FREQUENCY DISTRIBUTIONS OF MAXIMUM GROUND MOVEMENT CAUSED BY CUT AND COVER EXCAVATION



from Peck (1969)

Zone I

Soft to Hard Clay
Average Workmanship

Zone II

a) Very Soft to Soft Clay
  1) Limited depth of clay below bottom of excavation
  2) Significant depth of clay below bottom of excavation but N <6

b) Settlements affected by construction difficulties

Zone III

Very Soft to Soft Clay to a significant depth below bottom of excavation and with N>6

FIGURE 3 – SETTLEMENTS ADJACENT TO OPEN CUTS IN CLAY.

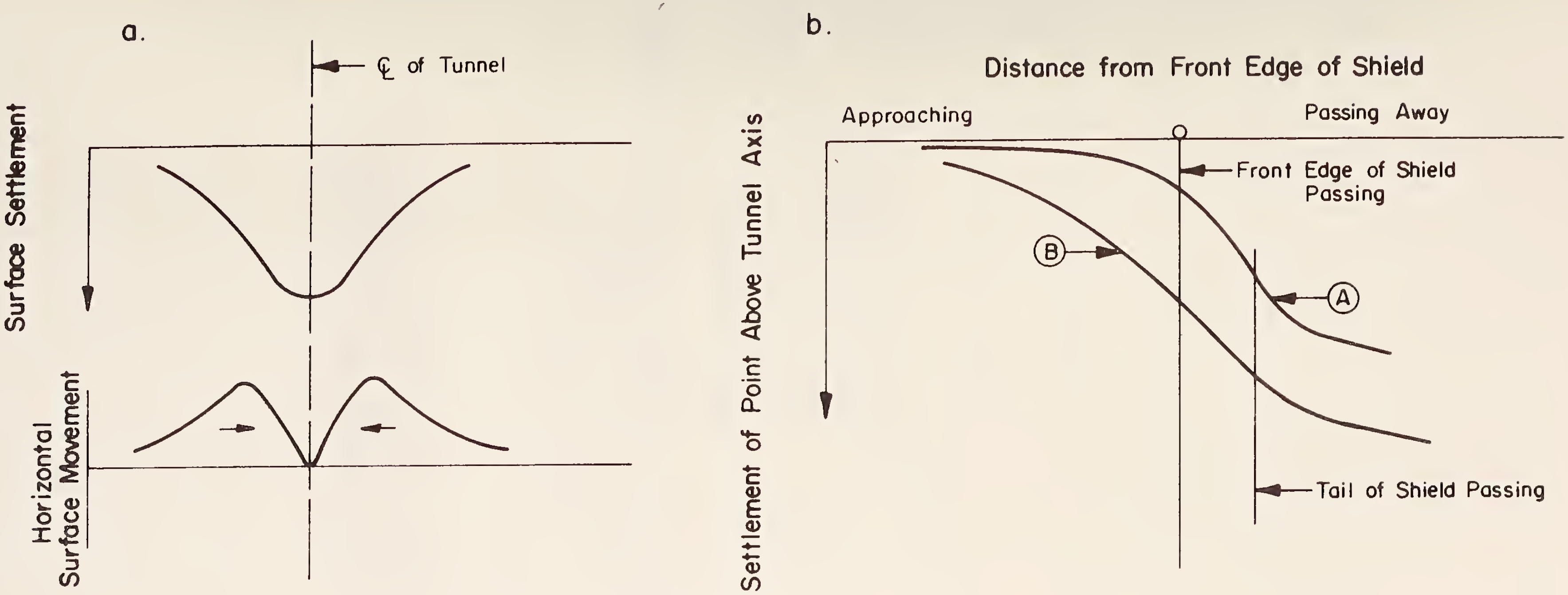

a.
℄ of Tunnel
Surface Settlement
Horizontal Surface Movement
b.
Distance from Front Edge of Shield
Approaching
Passing Away
O
Front Edge of Shield Passing
Tail of Shield Passing
Ⓐ
Ⓑ
Settlement of Point Above Tunnel Axis

FIGURE 4 - TYPICAL SURFACE MOVEMENT PATTERNS FOR SINGLE TUNNELS IN SOFT GROUND.

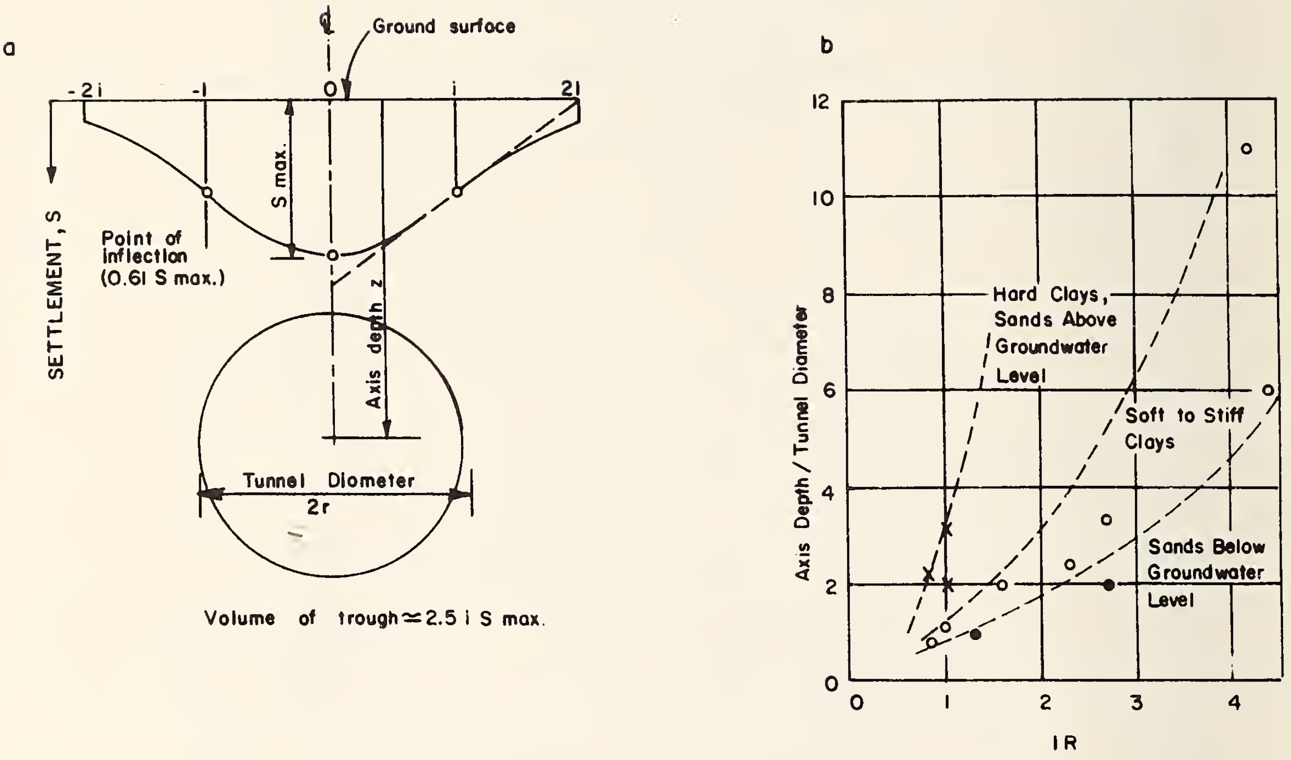

a
℄
Ground surface
-2i
-i
0
i
2i
SETTLEMENT, S
S max.
Point of inflection (0.61 S max.)
Axis depth z
Tunnel Diameter
2r
Volume of trough ≃ 2.5 i S max.
b
Axis Depth / Tunnel Diameter
0
2
4
6
8
10
12
Hard Clays, Sands Above Groundwater Level
Soft to Stiff Clays
Sands Below Groundwater Level
0
1
2
3
4
IR

FIGURE 5—PROPERTIES OF ERROR FUNCTION AND RELATION BETWEEN WIDTH OF SETTLEMENT TROUGH AND DEPTH OF TUNNEL FOR DIFFERENT SUBSURFACE CONDITIONS.

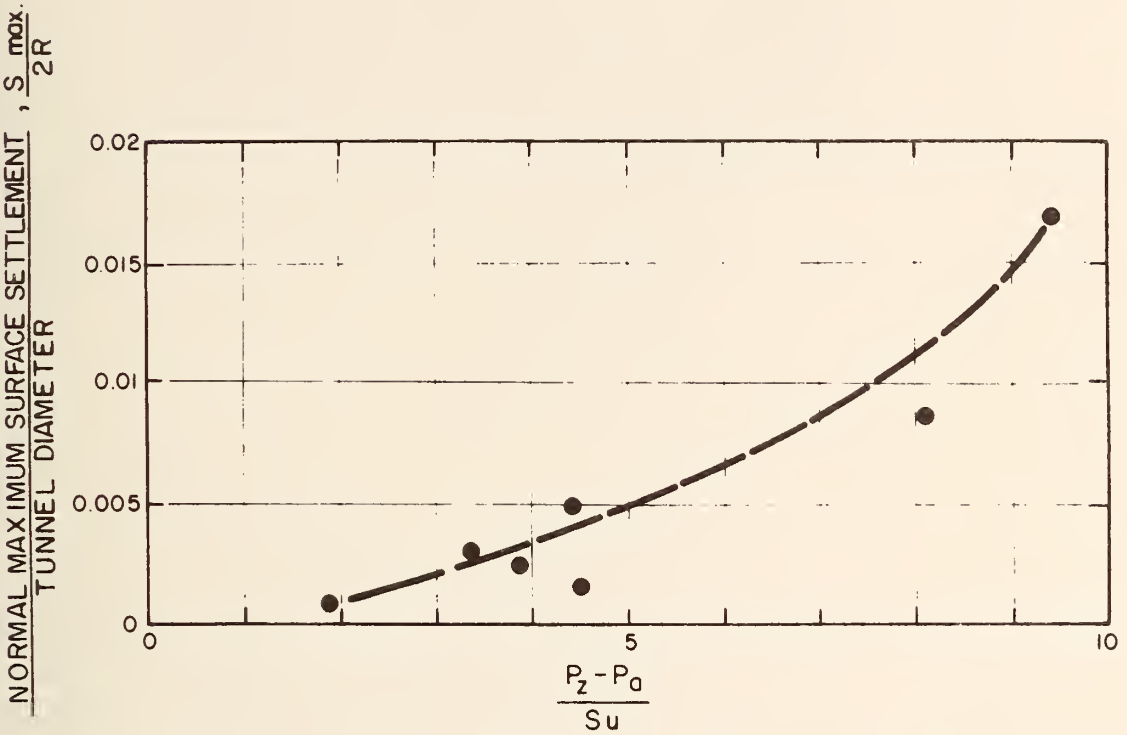


FIGURE 6–RELATIONSHIP BETWEEN $\frac{P_z - P_a}{Su}$ AND MAXIMUM SURFACE SETTLEMENT FOR TUNNELS IN PLASTIC CLAY.

TE 662
.A3 no.
FHWA-RD-74-57

U.S.
Re
of

BORROWER

Form DOT F 172

DOT LIBRARY
00054559

FHWA
R&D